AD 292 930

R 191

Technical Report

DESIGN FOR A CAST-IN-PLACE CONCRETE SHELTER

13 December 1962

U. S. NAVAL CIVIL ENGINEERING LABORATORY

Port Hueneme, California

DESIGN FOR A CAST-IN-PLACE CONCRETE SHELTER

Y-F011-05-328

Type C Final Report

by

J. R. Allgood, R. M. Webb, R. F. Swalley

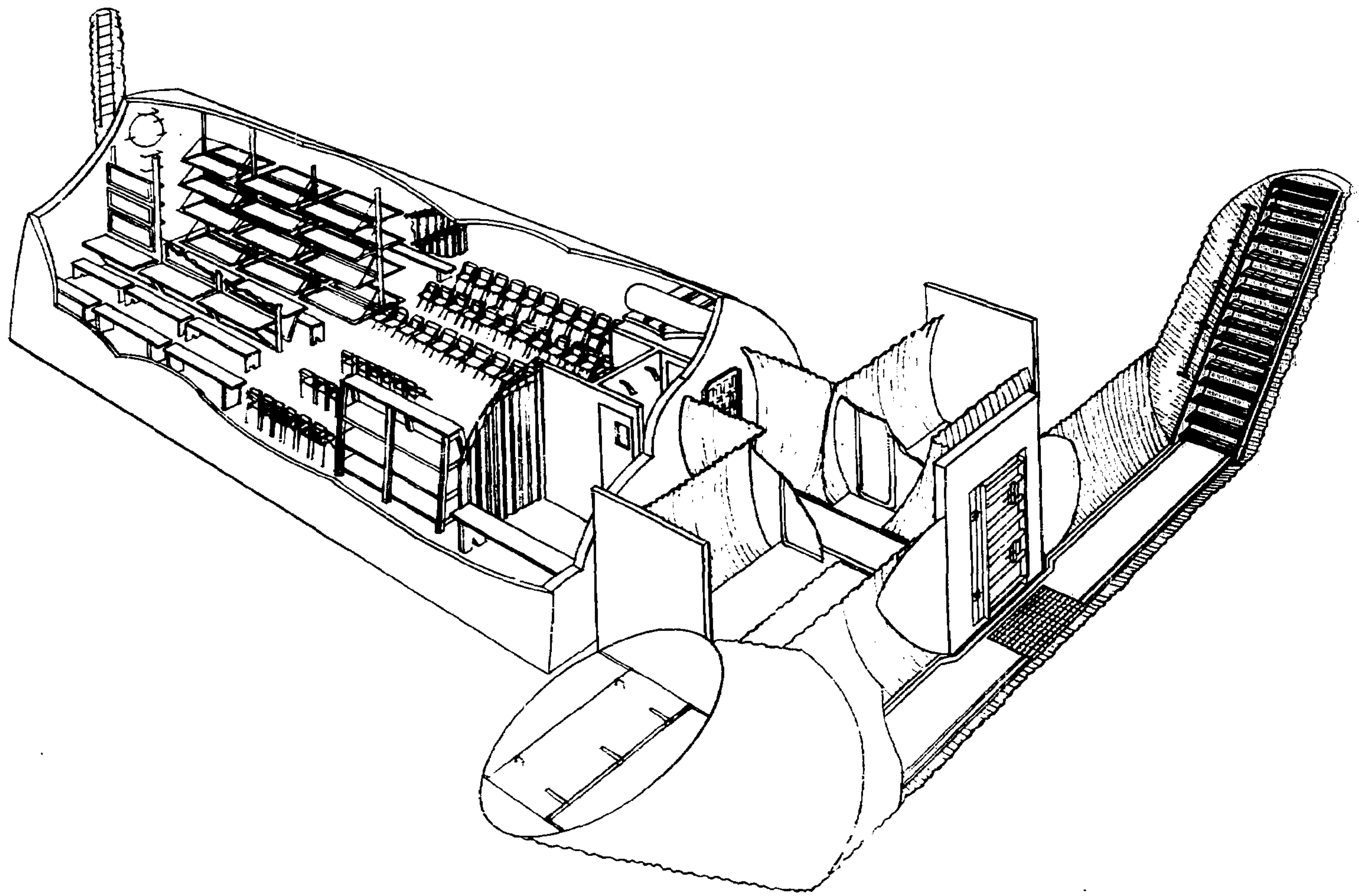
ABSTRACT

The objective of this task was to develop an economical, arch-shaped shelter, utilizing pneumatically-placed mortar, as an alternate to existing standard types. Criteria and plans for a 100-man shotcrete shelter are presented which will provide protection against an overpressure of 100 psi and concomitant effects from nuclear weapons. Shotcrete is recommended because of the economic advantages gained from using a single lightweight form as opposed to the heavy double form required for conventionally placed concrete. An effort has been made to provide a balanced and versatile design which may be adapted to the specific needs of various Commands.

Methods for the design of the basic structural components of the shelter are given, including a method for estimating the relative displacement between the floor and the foundation when the structure is subjected to blast loading. Simple yet adequate design procedures are given which are suitable for use in the design office.

Copies available at OTS $2.50
The Laboratory invites comment on this report, particularly on the results obtained by those who have applied the information.

CONTENTS



Cutaway view of cast-in-place concrete shelter.

INTRODUCTION

This final report presents plans and recommended criteria for a reinforced-concrete alternate to Navy standard personnel shelters. The work was accomplished under Task Number Y-F011-08-328, "Concrete Personnel Shelter." The purpose of the task, as defined by the Bureau of Yards and Docks,[1] was to "develop an economical arch-shaped structure, using pneumatically placed mortar (shotcrete), as an alternate to an existing standard type (of buried shelter) now included in NavDocks P-81."[2] The defined objective required that "plans and recommended criteria for a full-scale shelter be included as a part of the final report."[1] The shotcrete structure is desired to provide a variety of techniques and materials which can be utilized in the event of a national emergency.

The objectives of the task were pursued through a two-phase program, which included: (1) an investigation of techniques for forming and placing shotcrete, and (2) the development of plans, recommended criteria, and specifications for a shelter of shotcrete construction. The results of the first phase have been published.[3] They prove that concrete containing aggregates of at least 1/2-inch diameter can be placed by pneumatic means on lightweight single forms. The tests also demonstrated that surplus Quonsets are excellent forms for shotcrete shelters.

This report presents the results of the second phase of the program. It is based upon tests of buried metal and concrete arch shelters and studies of various aspects of the shelter design problem. Pertinent tests include those performed at the Nevada Test Site,[4,5] at the Eniwetok Proving Ground,[6] and in the Naval Civil Engineering Laboratory's atomic-blast simulator. Design information and studies from the literature have been heavily drawn upon and are referenced throughout the text.

It was not within the scope of this report to give a detailed treatment of shelter supplies and equipment; such treatment has been accomplished by others.[7,8] The intent is to treat in some detail the aspects of shelter design which are peculiar to the provision of blast and radiation protection in the 100-psi region and to limit treatment of other aspects of the shelter design problem to the minimum necessary for adequate accomplishment of the task objectives.

The material which follows includes a review of the general aspects of the shelter problem, an itemization of recommended criteria, and a presentation of an architectural layout. Methods for the design of the basic structural components are described and plans for the recommended shelter are provided. Specifications for the shelter have been prepared under separate cover.

SHELTER DEVELOPMENT

General Aspects of Problem

The emergency nature of disasters dictates that the philosophy underlying a set of criteria for a shelter be based on the tenet that survival is of primary importance and that luxuries are secondary in arriving at an economical shelter concept. Even with acceptance of this tenet, opinions will differ as to what are necessities and luxuries. For example, some will contend that other than a dirt floor in a shelter is a luxury while others will claim that a concrete floor is a small part of the total cost of the structure and is a necessity to the proper functional operation of the shelter. Obviously, such differences of opinion cannot be resolved here, hence they are left to those responsible for particular installations.

The plans and specifications developed in this study contain a degree of conservatism consistent with current knowledge of soil-structure interaction. The design is not, however, as conservative as some might think since most contemporary judgments are based upon the results of tests of structures subjected to kiloton weapons, which could produce results considerably different from tests of the same structures subjected to the same peak overpressure from megaton weapons. These differences must be recognized and it should be remembered that much work remains to be done to clarify the unknowns related to the design and habitability of underground structures. The development presented here, then, must be regarded as an interim solution to which refinements can be made as additional knowledge becomes available.

Requirements

Designs of military shelters will usually demand the provision of protection from high-explosive, nuclear, biological, and chemical attack. Shelter requirements for providing protection against these modes of attack depend upon the degree of protection desired. Unfortunately, no complete scientific studies have been made to determine the optimum degree of protection which should be provided; nor is there unanimity of opinion on this matter. It is necessary, therefore, to formulate a set of requirements based largely on judgment, keeping in mind that these requirements will likely be changed when the results of complete operations research and other studies and adequate laboratory and field tests become available.

In establishing the requirements it is well to recognize that no single shelter system can meet all of the needs of the Military Establishment. Simple fallout shelters may suffice in some cases, whereas deep-buried shelters may be required for sensitive installations. Nonetheless, shallow-buried structures offer a reasonable compromise for meeting many military shelter requirements. Such shelters can be readily adapted as personnel shelters, recovery-crew shelters, control centers, and equipment shelters with relatively minor changes in auxiliary and functional supplies and equipment. Although various types of shelters will have unique requirements, most differences will involve only shelter contents. The prime need, then, is for an economical shallow-buried structure which embodies a relatively high degree of protection and is readily adaptable to military needs.

These needs can be largely satisfied by shelters based upon the criteria given in the last column of Table I. The criteria require a resistance to a blast overpressure of 100 psi with concomitant resistance to radiation and other ABC effects. They were selected after a study of criteria, requirements, and specifications published by many investigators in the shelter design field and after due consideration of the basic problems involved.[9] Other requirements are reflected in the design drawings (Appendix E).

The summary in Table I presents most of the existing sets of shelter design criteria. Some of these are a result of actual tests, such as those conducted by the German Government at Waldrol,[10] the U. S. Naval Radiological Defense Laboratory at Camp Parks,[11] or the U. S. Army at Yuma.[12] Others are a result of studies in the United States which have not involved actual tests.[2, 13, 14, 15, 16, 17] Accounting for differences in purpose, the criteria of Table I provide a reasonably consistent pattern of what is needed in a shelter or shelter system. Doubt still exists, however, as to what constitutes an acceptable combination of minimum standards.

Discussion of Criteria

Occupancy Time. An occupancy time of 14 days is almost universally accepted, although there are arguments for a 7-day period.[15] In most situations (except when cobalt bombs are used) the radiation level will be reduced sufficiently after 14 days to permit movement of personnel to a remote locale with a tolerable radiation level. A biological attack might require an extended stay period, but a 14-day stay time is considered expedient for design purposes. It would be possible to leave the shelter for short periods of time during the 14-day stay for operational purposes.

Personnel Capacity. Studies of the psychological and control problems encountered in shelters cite the advantages of maintaining the capacity of shelters at 100 or less persons. Two factors favor smaller, more widely scattered shelters. First, access is readily available for more people. Second, psychological problems are less likely to arise when people are not crowded together.[18] On the other hand, large shelters are less expensive per capita. From the military standpoint it would seem desirable to have a number of smaller, dispersed shelters to better insure survival. A shelter capacity of 100 men is considered a suitable compromise of all factors. Provision should be made for an emergency overload capacity of 100 percent, with additional blast-protection capacity in the entranceway.

Overpressure Resistance. For military purposes it is usually desirable that shelters be capable of withstanding a relatively large blast overpressure loading. A resistance of 100 psi provides a high probability of survival from kiloton and megaton blasts,[18] although a recent study suggests that resistance to 250 psi can be justified.[19] No doubt resistance to 1,000 psi can be justified for missile complexes and similar protective structures, but such installations are beyond the scope of the type considered here.

It is almost axiomatic that an economical shelter must be buried to resist 100 psi; however, the depth of burial is usually determined by the amount of radiation attenuation necessary rather than by the overpressure.

Radiation Protection. Radiation protection is provided economically by earth cover. At a range corresponding to 100 psi the maximum initial gamma radiation would be 1×10^5.[20] A radiation reduction factor* of 10,000 is necessary to reduce this amount of initial radiation within the shelter to 10 roentgens. A depth of cover of 5 feet and an arch thickness of 10 inches (concrete) combine to give an equivalent depth of 6 feet 3 inches. This amount of cover (based on 103-pcf soil) will provide the required reduction of initial gamma radiation.[21] Negligible initial neutron radiation will enter the shelter through the ground under the worst conditions at a range corresponding to 100-psi overpressure. The effect of residual radiation will be negligible with 6 feet 3 inches of effective soil cover, so prompt radiation is essentially all that will be received by occupants during the shelter stay period.

Entrance Time. The allowable time to fill a shelter depends on the warning time in advance of an attack. A warning time of 15 minutes is considered maximum in this day of high-speed intercontinental missiles. For design purposes a warning time of 10 minutes is selected because of the probability of increased missile speeds and delivery from submarines.

Entrance and Exit. The entrance and emergency exit should have a configuration that will cause the immediate radiation level to be reduced to 10 roentgens at the opening in the arch endwall. For best radiation protection, minimum length of entrance tunnel, and minimum cost, the stairway should be as steep as practicable and approximately 3 feet wide. For these conditions at high densities (over 0.2 persons per square foot) the velocity for hurried movement would be about 2.2 feet per second on the stairs, and slightly greater in the level passageway.[22] The fill time for a typical entranceway with stairs for 100 persons moving at 2.2 feet per second and at a flow rate of 60 persons per minute (single file) would be 2 minutes. A minimal 8-minute period would be left for moving to the shelter and closing the doors.

* The radiation reduction factor is the ratio of the radiation dosage which would be received exterior to the shelter to that dosage which would be received inside the shelter.

Table I. Personnel Shelter Criteria

Source Reference No.	P-81 2	OCDM 13	NRDL 11	Panero 14	Sub	Army (Yuma) 12	German* (P-290) 6	AEC 16	German (Waldrol) 10	Voorhees et al 17	Proposed
1. Time of Occupancy, days	14	14	14	14	-	1	-	14	5	14	14
2. Capacity, persons	50-100	varies	100	varies	-	40	50	100	45	-	100
3. Overpressure Resistance, psi	varies	-	varies (10-35)	-	-	60	125	35	125	-	100
4. Radiation Reduction Factor	-	5,000	-	5,000	-	-	-	10,000	-	-	10,000
5. Entrance Time, minutes	-	-	-	-	-	-	-	5	-	-	10
6. Means of Ingress and Egress, no.	-	2/200	1/100	2	-	2/40	2/50	-	2/45	-	2/100
7. Space: Area, ft^2/man Volume, ft^3/man	10 -	10 65	12 115	16.5 -	- -	17.1 90	about 8 about 88	12 117	6.0 about 50	12-20 -	7-8 65
8. Air Supply, cfm/man	-	3	6.75 min.	4	-	5	-	-	5.9	3	6**
9. Emergency Air Supply, cfm/man	-	-	CO_2 bot. CO_2 absorp.	-	-	-	yes	-	1.2	-	2
10. Air Purification, type	Filter	Commerc. filter	-	Dual filters	-	-	Sand filter	-	Sand filter	Glass-fiber filter	Collective protector
11. Blast Closure	yes	no	no	no	-	no	no	no	no	no	yes
12. Decontamination Showers, no.	yes	1/200	no	no	-	no	no	no	no	no	1/50
13. Lavatories	1/20	1/50	Bucket	4/144	1/15	-	-	-	-	-	1/100
14. Toilets: Type Number	Chemical 1/25	Flush 1/35	Chemical 1/50	Chemical 4/144	Flush 1/20	Chemical 1/40	Chemical 2/50	Flush 1/25	both 2/45	either 1/35	Chemical 1/50
15. Potable Water, gal/man/day	adequate	1	0.50	4	1 (act.)	-	-	3/57	-	0.25	0.50
16. Food, ft^3/man/14 days	Army 1A	2	varies	3	28	-	-	-	2,000 cal/man/day	-	2 (Army combat ration)
17. Lighting: Type Intensity	Fluor. -	- 5-25 fc	Suffic. to read	Incan. 2-25 fc	- -	Fluor. -	- -	- -	Fluor. -	- 5-25 fc	Fluorescent 10-20 fc
18. Emergency Lighting	-	-	-	Trickle lanterns	-	-	-	-	-	Flashlt & Candles	Lanterns, Candles, Phos. tape
19. Sleeping Facil., % of cap.	33%	50%	100%	50%	-	-	-	50%	33%	-	50%
20. Trash Disposal, type	-	yes	Polyeth. bags	-	-	-	-	-	-	-	2 Cl cans, Plastic bags
21. First-Aid Equipment	-	yes	yes	-	-	-	-	-	-	yes	0.5 ft^3
22. Fire-Fighting Equip., type	-	yes	-	-	-	-	-	-	-	-	2 CO_2 bottles
23. Rad. Monitoring Equipment	-	yes	Dosimeter	-	-	-	-	-	-	-	2 AN-PDR-2, 1 AN-PDR-10, 1 AN-PDR-18, 10 Dosimeters
24. Commun. Equipment, type	-	Receiver, Telephone	Transceiver, AM receiver	-	-	-	-	-	-	Battery radio	1 Transceiver, 1 AM receiver, 1 Telephone

* These cases are for Class A, the largest type.
** May vary upward to suit geographic location of shelter.

Care must be taken in selecting the entry and exit configurations to prevent excessive radiation streaming into the shelter and to preclude the occurrence of large reflected pressures on or near the door. The exit problem is solved most easily by making it of conduit filled with dry material such as coarse washed gravel. The gravel may be drained into the shelter when it is necessary to use the emergency exit.

Space. The floor area and the cubic content of a 100-man shelter depend primarily upon the amount of sleeping and seating facilities required. Areas from 3.55 to 17 square feet per person are specified in Table I. The German criterion of 3.55 square feet of area and 27 cubic feet of volume per person[54] is considered an absolute minimum for survival. An area of 7 to 8 square feet with a volume of 65 cubic feet per person is recommended as a more comfortable minimum, which would provide for some overload capacity of the shelter. The volume of 65 cubic feet per person is considered a minimum compatible with the recommended floor area.. If beds are provided for bunking no more than 50 persons at any one time, this area-volume would allow an emergency overload of 100 percent. It is emphasized that an area of 7 to 8 square feet per person can be justified only if sufficient volume is provided and if operational considerations are not of such a nature as to be the controlling factor.

Air Supply. The preceding space requirements presume a ventilated shelter. An adequate supply of fresh air must be maintained to restrict effective temperature rise, remove offensive odors, and provide necessary air renewal. The quantity of air required cannot be readily defined because of its dependence on odor control, humidity, and the outside air temperature.[23] For outside conditions of 75 degrees dry-bulb and 65 degrees wet-bulb, the standard 600-cfm U. S. Army Chemical Corps filter unit[24] will maintain an effective temperature of about 80 degrees Fahrenheit within the shelter. For more severe outside conditions a unit of considerably greater capacity, and possibly air conditioning, may be required particularly if a shelter overload is anticipated.

Technical personnel associated with tests of a Navy shelter at Bethesda, Maryland, consider 2 cubic feet per minute to be adequate for controlling the O_2, CO, and, CO_2 content of the air, but that larger quantities will usually be required to maintain the effective temperature at an acceptable maximum. In warmer climates it is likely that air conditioning will be required to limit the maximum effective temperature to less than 85 degrees. There are still uncertainties concerning shelter ventilation requirements and future tests may indicate a need for modification of requirements which are presently regarded as satisfactory.

Care should be taken to assure that a positive pressure is maintained within the shelter to prevent contaminants from entering. Generally a pressure of 1/2 inch of water is adequate for this purpose. Experience has shown that even this small pressure is difficult to maintain unless great care is taken in the design and construction of the shelter to eliminate any possible source of leaks. The exhausted air should be drawn through the decontamination areas to assist in the removal of contaminated matter and to assure that such matter is not carried into the living area. Vents which can be manually opened and closed (so as to permit maintaining a relatively constant internal pressure regardless of the air volume input of the ventilation system) can be placed in the shower doors. A monometer should be installed for measurement of the pressure within the shelter.

Blast-closure valves are required for both the air inlet and exhaust ducts. The closure valve should be constructed so that leakage is minimal, thus eliminating the need for a surge chamber. A spare filter for the filter unit should be stored on top of the main unit in the event the filter is damaged through malfunction of the blast-closure valves.

Ideally, hand-blowers with a minimum capacity of 300 cfm should be attached to the intake ventilation system to maintain life in the shelter if the power fails. The hand blowers should be connected to the intake system in such a way that the filter of the filter unit is utilized. In extreme emergencies, the air could be provided by by-passing the filter unit or by opening the exterior doors. Of course, such measures would risk contamination of the interior of the shelter and would provide air only for a short period of time unless atmospheric conditions were quite favorable.

Decontamination. In a military shelter it would be necessary to provide showers for decontamination if BW or CW or severe fallout is anticipated and if entry of contaminated persons is expected. Two showers are desirable — one for preliminary washdown while clothed and one for washdown when undressed. The preliminary washdown can be accomplished in the shelter entry. A simple gravity tank arrangement, with a settling tank below for purification, would allow a recirculating water system. Chlorine concentrate, which is a sterilizing chemical and an effective destroyer of certain biological agents, should be available. [8]

Sanitation. It is desirable to keep the number of toilets to an acceptable minimum because of the relatively large amount of space they require. One chemical toilet for 50 persons is considered sufficient if portable toilets with plastic bags are available for emergencies. Portable toilets can be set up in the shower or the entryway to afford privacy once the shelter is closed. Chemical toilets are preferable to water closets for the following reasons:

1. Lower cost. —Elaborate piping is not required.

2. Reduced water-storage capacity. —No water need be stored to flush toilets.

3. Added safety. —The openings in the shelter necessary for water closets are not needed.

If water closets are used they should be equipped with check valves to prevent waste from being forced back into the shelter by the blast.

Potable Water. Man can survive on 1 quart of water a day when inactive. Considerably more water should be available, however, because of the possibility of high interior temperatures and the probability of intermittent outdoor or indoor activity. Two quarts per man per day is considered an acceptable minimum in mild climates, but where shelter occupants are subjected to relatively high effective temperatures up to 1-1/2 gallons per person per day may be necessary. Sufficient water should be provided for minimal accommodation of 100 percent overload.

Water may be stored interior or exterior to the shelter. Interior storage has the advantage of not requiring openings through the shelter for pipes and of assuring against loss of water through a failure in the supply system caused by the relative motions between the soil and the shelter and the soil and the storage tank. Interior storage does require considerable cubage which might be used for storage of other supplies.

Perhaps more important than whether the water is stored inside or outside the shelter is whether a circulating supply system should be used. Water can be stored for long periods of time in proper containers without loss of purity. Thus, a circulating system is desirable but is not necessary to the sustenance of life.

From a strict safety and minimum cost point of view, interior storage in sealed containers is recommended.

Food. It is desirable to have a type of food which requires a small storage cubage and which will not deteriorate rapidly with age. The individual combat ration used by the Army meets both requirements. It contains 3,667 calories, far more than the 1,500 to 2,000 minimum daily requirement of an adult. Storage of Army rations for a 14-day stay time requires 2 cubic feet of space per person. Additional information on "hotel packages" is available elsewhere.[7, 18]

Lighting. Lighting facilities should be minimal to help maintain a tolerable heat level. A light intensity of 20-foot candles is sufficient in sitting areas, and 10-foot candles would be adequate in entranceways. No lights at all are required in sleeping areas, although minimal lighting may be provided for convenience. Five-foot candles are adequate in all areas outside of the sitting area. Fluorescent lights give off less heat than incandescent and should be used in preference to the latter.

Several long-life fluorescent lanterns should be available for emergency purposes. Also, phosphorescent tape should be placed around doorways and at partition corners to define openings.

Sleeping and Seating Facilities. Tiered, movable, stretcher-type bunks are inexpensive and most suitable for shelter use. Only 50 bunks are required if sleeping is done in two shifts.

Folding chairs are desirable for seating since they can be folded and stored when the space is desired for other purposes. Chair seating should be supplemented with benches.

Trash Disposal. Trash would consist primarily of contaminated clothing and refuse from the food rations. Two garbage cans, located in a radiologically isolated area, are adequate for trash-disposal purposes. Presumably these cans could be emptied periodically during the shelter stay period if necessary.

Fire Protection. Threat from the fire storm which may accompany a nuclear attack depends largely upon the location of the shelter. In most military installations the problem would not exist, but in a few cities in the U. S. it might be a problem. Still, the threat must be considered. The major effect of a fire storm is to consume oxygen in the air exterior to the shelter and to develop a partial vacuum which may draw air from the shelter. A warning should be posted within the shelter that if a fire storm develops the shelter should be kept sealed for 5 to 6 hours. The carbon-dioxide content would build up to about 6 percent, and the oxygen concentration would reduce to 15 percent; however, these conditions are not deleterious, as submarine studies[26] and tests on rats[27] have shown. During the 5- to 6-hour period, the fire storm would have subsided sufficiently to allow use of exterior air. Thus, the problem of a fire storm can be solved without artificial supply of oxygen or absorption of carbon dioxide. The effective temperature, however, may rise to an uncomfortable level unless air conditioning is available. In cases where a seal-up of longer than 6 hours is envisioned, a complete carbon-dioxide absorption unit and oxygen-introduction system should be added to the existing air-supply system.

Two small carbon-dioxide fire extinguishers should be provided to combat fires within the shelter. Wherever possible, materials within the shelter should be fireproof.

Radiation Monitoring Equipment. Table I indicates the minimum amount of radiation monitoring equipment that should be in the shelter. Additional equipment may be included to meet military operational requirements.

Communications Equipment. Communications equipment requirements also depend upon the military function of a particular shelter. Table I indicates the minimum desirable.

Electrical Equipment. Power requirements will vary with the operational requirements of a shelter. For a personnel shelter, the only power required is 3 kilowatts for the filter unit, lighting, and the communications equipment (providing no electric coffee urns, hot plates, etc., are used). Thus, a 5-kw generator will suffice if procedural precautions are taken. A 5-kw generator cannot meet the demand of the fixtures and simultaneously provide sufficient power to start the motor on the filter unit. Therefore, a 5-kw generator will suffice only if precautions are taken to assure that other loads are disconnected for the brief time during which the motor is started. Normal line power should be available for non-emergency operation.

Concern has been expressed that generators of less than 15-kw capacity are not sufficiently reliable to use in a shelter. In the interest of conserving fuel and minimizing air requirements, those who are apprehensive about the reliability of small generators might secure two of identical model, use them intermittently, and cannibalize one for parts if both should break down. It is doubtful that this would be necessary since emergency air and light sources can be activated in the event of a generator failure. In addition to the larger initial investment, use of large generators involves problems of providing space, fuel, and air to meet their needs.

Where two or more shelters are nearby, a central large diesel generator, suitably arranged for self starting, could be tied into the several individual electrical systems with single small gasoline generators in each shelter retained as standby units. The several small generators could be cross-connected to provide back-up power in the event of failure of any single unit.

The generator should be wired to cut off automatically when the blast-closure valves are activated if the valves are of a design which cannot be relied upon to open when the pressure wave has passed. Shutting down the generator will prevent an accumulation of exhaust gases. Valves such as the recently tested (but not yet reported) Breckenridge valve eliminate the need for such shutdown.

Miscellaneous Equipment. Other items that should be provided are extra clothing, hand tools, and adequate medical and first-aid supplies. Optional items such as periscopes or games might be included; however, in general there will not be sufficient space within the shelter to accommodate much extraneous equipment unnecessary to the sustenance of life.

Many items, such as types of electrical conduits and water seals, are not discussed here nor included in Table I. Only those basic items are included which are considered essential for the conceptual design of a military personnel shelter.

Architectural Layout

A shelter which meets the criteria of Table I as discussed above is shown in Appendix E. Many layouts are possible within the imposed limitations. The authors believe the layout shown is among the more functional.

Selection of an entry configuration is the most perplexing problem in the layout of a shelter. There are numerous considerations in arriving at a suitable layout, many of which are not obvious at first thought. An arrangement must be sought which will satisfy all of the various requirements, including:

1. Short ingress time

2. Provision of blast protection

3. Sufficient radiation reduction

4. Access to the generator and filter unit from within the shelter for repair and maintenance

5. Isolation of the shelter living area from the noise and fumes of the generator

6. Low cost

Several of these requirements are incompatible, particularly short ingress time and low cost.

The cheapest and simplest entranceway or emergency exit would be a vertical tube with a ladder as shown in Figure 1. For trained military personnel, the flow rate on such ladders is 17.4 persons per minute, [22] which would allow the shelter to be filled in 6 minutes. For civilians or untrained military personnel, a flow rate of 5.6 persons per minute is found to be typical, [22] which would allow the shelter to be filled in 18 minutes. The latter situation is considered typical for shore establishments where many civilians and untrained military personnel would be involved. The fill time of 18 minutes does not meet the criteria of Table I, hence the vertical ladder cannot be employed unless one is willing to relax the requirements for fill time. Multiple vertical tubes could be used to reduce the fill time, but with such a configuration the cost would approach that for other layouts which provide much easier access.

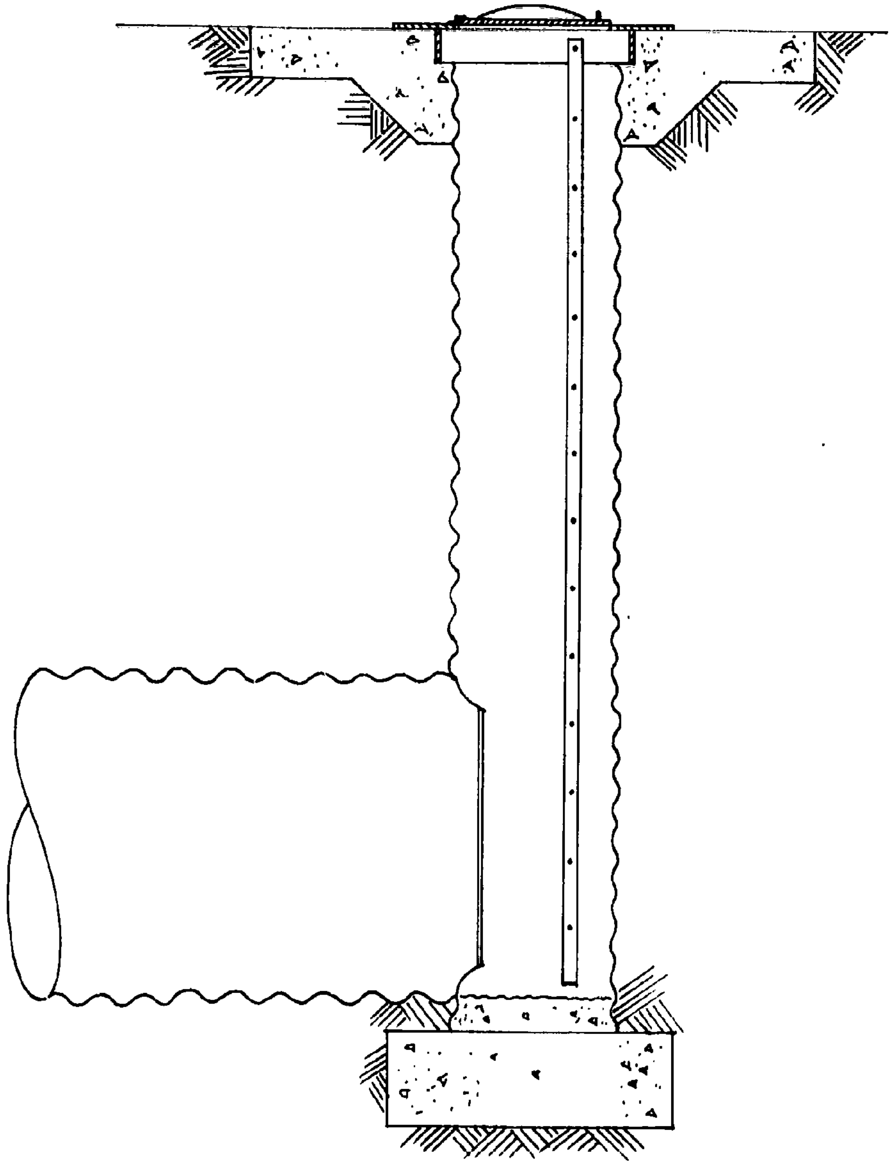


Figure 1. Vertical tube as an entry.

Perhaps the next layout one might envision is the L-shaped configuration shown in Figure 2. This would be a very excellent entry for low overpressure regions, but it poses problems for the high overpressure regions. Specifically, the reflected pressures at the end of the tunnel and at the door are exceedingly high. For the 100-psi level, the reflected pressure from a shock directed down the tunnel is on the order of 500 to 800 psi.[28] Providing a tunnel endwall and a door which will resist this pressure becomes an expensive problem. One might suggest the use of a horizontal door at the surface to overcome the problems of reflected pressure in the L-shaped entry. This would seem to be a reasonable suggestion, but when one actually gets into the design of such a door it is soon discovered that the mass of the door becomes exceedingly great. Consequently, the door becomes difficult to open and close, a massive foundation is required, the probability of jamming increases, and the cost goes up.

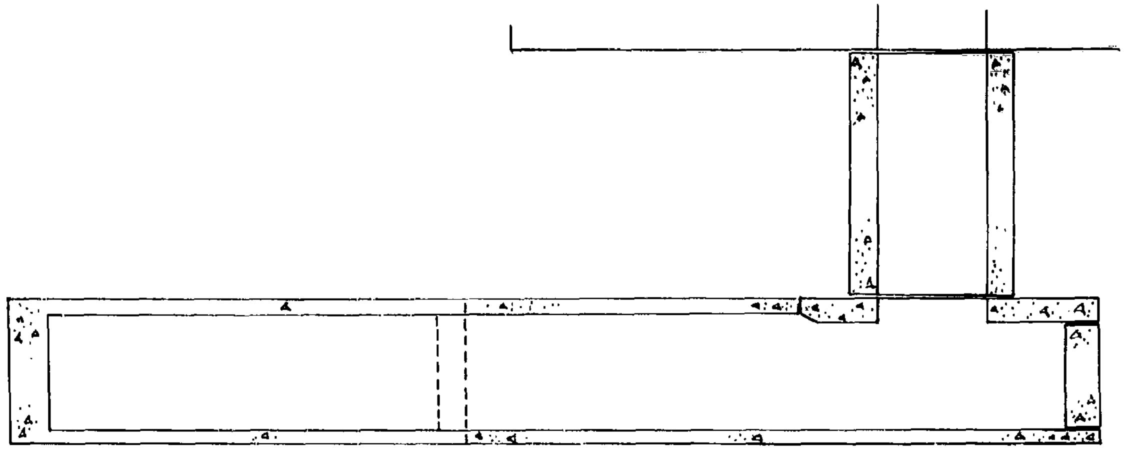
In the proposed alternate entry, shown in NCEL Drawing No. 936971 of Appendix E, the generator and filter unit are located in rooms within the main shell of the shelter. This arrangement is satisfactory provided that large lateral motions of the arch do not occur on loading. If lateral displacement occurs, the partitions will be cracked and the filter-unit and generator rooms will no longer be sealed from the living area, and carbon monoxide or gasoline fumes might get into the shelter. For these reasons, the alternate entry, while much more aesthetic in appearance, is not recommended over the prime entry.

The prime entry, Drawing No. 936965, meets all of the requirements listed above except one — it is expensive. Unfortunately, meeting the established criteria is not readily accomplished inexpensively. Steps and handrails are omitted from one side of the tunnel to cut costs; nonetheless, the entry represents a sizable percentage of the total cost of the shelter, as is revealed in more detail later in the report.

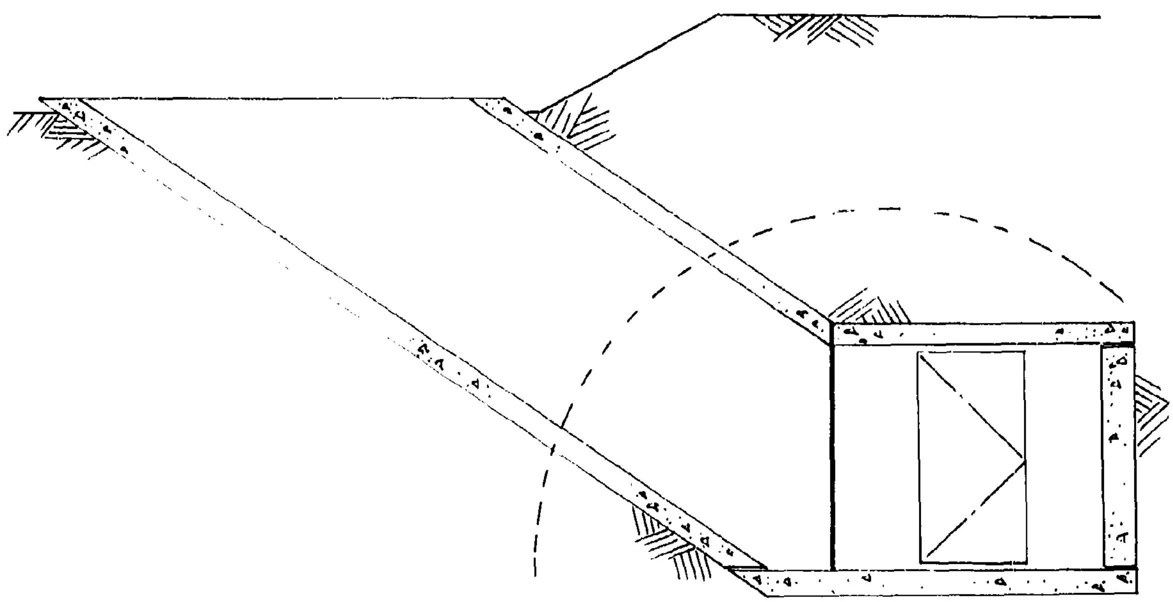
The plan of the shelter proper, Drawing No. 936961, provides for sleeping 48 men at one time (two less than suggested by the criteria of Table I) and seating 52 others on chairs. Additional seating is available on the benches in the bunking area and in the dress and dry area. An open area approximately 11 feet by 12 feet is available for exercising, food distribution, and other miscellaneous functions. Reasons for the layout become more apparent after a discussion of the functional characteristics of the shelter.

Functional Characteristics

The flow of traffic on ingress may be directly into the shelter or, at the discretion of the shelter commander, through the decontamination showers. A preliminary shower is located in the entry which is also intended to serve as the undress area. GI cans for contaminated clothing are located inside the filter unit room. After passing through the showers one enters the dress and dry area which is screened from the living area by a curtain. Decontaminated persons would then receive a smock from the storage area and proceed into the shelter living space.



Plan



Section

Figure 2. L-shaped entry.

The berth arrangement is such that the lower berths can be converted into tables for special occasions. Folding benches are provided for use at the tables and for auxiliary use in the event of overload of the shelter.

Two chemical toilets are located within the shelter and two portable lavatories with disposable plastic bags are to be provided for emergency use. The air-supply system is arranged so that odors from the toilets will be purged into the exhaust air.

Flow of air within the shelter is as follows: Exterior air is drawn in through the filter unit, which removes the contaminants; it is then distributed within the shelter and exhausted through the decontamination areas and generator room to the atmosphere. The doors to the filter unit room and the generator room are sealed to insure the desired circulation of air and to obviate the possibility of gases and fumes from the generator entering the shelter. These rooms also serve as surge chambers in the event of leakage past the blast-closure valves.

The filter and the generator are both accessible in a protected area from within the shelter for servicing and maintenance. Still, the filter unit is isolated from the shelter so that radioactive particles accumulated in the filters cannot contaminate persons within the living area.

Other functional features of the shelter are relatively standard.

DESIGN OF SHELTER

Design Parameters

Most of the design parameters are implicit in the requirements of Table I. Others are given in Table II. The site conditions listed in the latter table are restrictive, but insufficient information is available at present to warrant liberalizing or generalizing them. The problem of water tables close to the surface is a particularly difficult one which requires careful attention.

Assumptions which are used in the design of the shelter are given in the sections of the report to which they apply. Rather gross assumptions are necessary because of insufficient knowledge in certain areas. Knowledge of soil-structure interaction is at present particularly limited, although extensive research currently in progress should alleviate the situation.

Table II. Design Parameters

Description	Symbol	Units	Value or Equation	Comments
Concrete				
Compressive strength of concrete at age of 26 days unless otherwise specified	f'_c	psi	4,000	Type III cement 7-day strength
Bond		psi	$0.15\ f'_c$	
Shear: Ultimate shear stress for members with web reinforcement		psi	$0.04\ f'_c + 4{,}000\ p + rf_y$	$p = A_s/bd$ $r = A_v/bd$
Ultimate shear stress for members with no web reinforcement	v_y	psi	$0.04\ f'_c + 4{,}000\ p$	
Modulus of elasticity	E_c	psi	$1{,}800{,}000 + 460\ f'_c$	
Steel				
Dynamic yield stress		psi	$1.25\ f_y$	
Modulus of elasticity	E_s	psi	30,000,000	Static value and assumed dynamic value
Soil				
Soil density		lb/ft^3	120	In-place density of sand at the site: Backfill consolidated by vibratory methods or by water
Foundation modulus	k_o	$lb/in.^3$	258	
Modulus of passive pressure	K_p	$lb/in.^3$	174	
Ground-water table	G.W.	feet	3 feet or more below bottom face of footings	
Allowable (static) soil-bearing pressure		psf	6,000	
Angle of internal friction	φ	degrees	35° (minimum)	Static value
Overpressure				
Peak value of long-duration transient overpressure at the site	P_o	psi	100	
Load Factors				
Arch flexure		unitless	1.0	
Arch shear		unitless	1.5	
Endwall		unitless	1.5	
Entryway		unitless	1.5	
Foundation		unitless	1.0	

Behavior of Buried Arches

Before proceeding to a discussion of the design of the shelter it is instructive to review the general aspects of behavior of buried shelters subjected to long-duration blast loading. As the pressure wave travels downward through the soil at the seismic velocity (about 1 foot per millisecond in dry, well-compacted sand) it rapidly envelops the structure. The nature of the response is such as to produce essentially radial loading on the arch in a relatively short period of time. Thus, the only moments of consequence are those induced in the first few milliseconds after the pressure wave strikes the surface of the arch. Radial loading occurs primarily through the development of passive pressures on the sides of the arch as they move into the soil. It is the passive pressures which limit the moment and make it possible for a flexible structure to sustain large blast loads.

The motion of a buried arch to a long-duration blast is fundamentally a body motion resulting from compaction under the footings and punching of the footings into the soil. This action is desirable if the footings are designed so that the punching is not excessive. The body motion of the arch into the soil assures the development of the full arching capacity of the soil. Resistance from arching, incidently, is relatively insignificant for shallow-buried structures as compared to that from the development of passive pressure.

At present the design of buried arches is based upon an assumed equivalent-static loading because of limitations in the knowledge of the behavior of soil-structure interaction and the availability of suitable analytical methods. The equivalent loading must be based on the judgment of the engineer and the limited amount of test data available. [5]

Arch Design

Of various design techniques proposed for buried arches, [29] perhaps the most elementary and satisfactory is the plastic design method. [30] This method involves assuming the location of plastic hinges and solving a virtual-work equation for the plastic moment, from which the section properties may be obtained. Results from this method and others, of course, are contingent upon the correctness of the assumed configuration of loading.

Test results indicate that the space-time variation of loading on a buried concrete arch is approximately as defined in Figure 3. At the time of maximum deflection, the load distribution on the arch is given by the relation

$$p = p_o\left(1 - \frac{3}{4}\cos\theta\right) \tag{1}$$

This, then, is the load used as the equivalent static load for designing against long-duration blast loads. A design using the plastic method and based on Equation 1 is given in Appendix A.

Equation 1 does not account for the slight asymmetry of loading noted in the tests and, therefore, it is recommended that equal tension and compression reinforcement be used to avoid failure at plastic hinges from lateral motions of the structure. Also, Equation 1 does not allow for any reduction in pressure with depth or allow for reduction in peak overpressure due to the transient nature of the loading.

Tests of small-scale buried arches which have been performed in the NCEL blast simulator show that there is a great deal of difference between the behavior to short- and long-duration loading. The increased duration at 100 psi from a megaton weapon over a kiloton weapon is exceedingly important in influencing the resistance level of a buried arch. The influence of load duration on buried structures has been displayed in chart form based on an approximate analytical development.[31] From the chart it is readily determined that for a cubical structure with a floor area of 1,000 square feet, the ratio of the peak blast load which the structure will withstand to the static collapse load is 2.6 for a 20-kiloton weapon and 1.4 for a megaton weapon. That is, the roof would have to be almost twice as strong to resist 100 psi from a megaton weapon as it would to resist the same pressure from a 20-kiloton weapon.

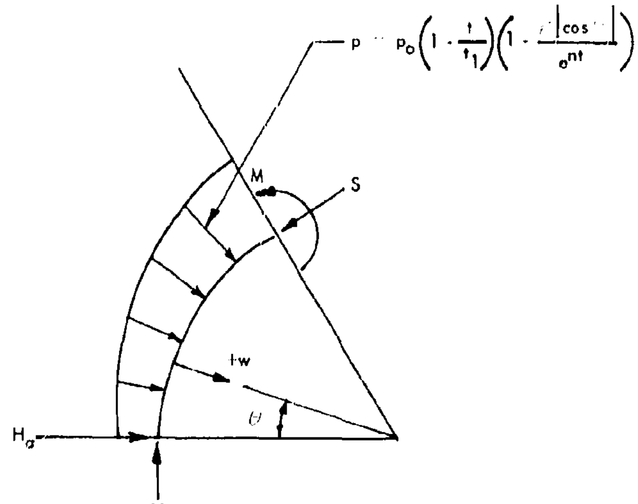


Figure 3. Load distribution on arch.

Test results[4] and shell theory show that tensile stresses develop at approximately 45 degrees to the longitudinal axis near the endwalls. Steel must be included to prevent excessive cracking in these regions. These tensile stresses are, in effect, diagonal tension stresses when the entire arch is considered as a beam. Design of the necessary diagonal tension steel is given in Appendix D.

It is desirable that the arch steel descend downward into the endwalls to assure continuity between the arch and the endwalls. The arch steel should also be tied to the footing reinforcement.

Endwalls

Design of the endwalls is facilitated by use of the yield-line theory[32] with a procedure for determining the dynamic response of simple beams. Assuming that the endwall acts as a semicircular plate with fixed edges and that the effect of axial compression in the endwall is negligible, a yield-line pattern is developed as shown in Figure B1 of Appendix B. The corresponding relationship between the unit moment, m, and the applied uniform load, w, is of the form

$$m = c_s w$$

where c_s is a constant.

The load on the "equivalent simple beam" (of span L equal to the endwall radius), is computed from the known ratio of moments:

$$w_{bm} = \frac{8m}{c_s L^2}$$

With this load, one can enter available design charts[33] to determine the percentage of tension and compression steel required to resist a given blast load. The procedure is delineated in Appendix B.

The portion of the overpressure which acts on the endwall will depend on the characteristics of the soil and may be calculated from Rankine's Equation.[34] If the angle of friction of the soil is not readily available, the following ratios of lateral pressure to ground-surface overpressure may be used.[30]

Cohesionless soils, damp or dry	k = 1/4
Cohesive soils, not saturated	k = 1/2
Cohesive soils, soft consistency	k = 3/4
Saturated soils	k = 1

Reinforcing steel should be placed normal to the yield line wherever possible and care should be exercised to ascertain that the arch reinforcement extends beyond the high-moment regions of the endwall.

Entranceway Design

Tube Layout and Design. The entranceway (Drawing No. 936965, Appendix E) consists of elliptical corrugated-metal tubes in the shape of a "T," which allow ingress and egress. Small compartments are provided off the leg of the "T" for the generator and the collective protector. A blast door and supporting wall are located at the intersection of the legs of the entry. This geometrical arrangement was chosen to provide isolation of the shelter occupants from radiation, blast, and fumes, as discussed under the section on requirements.

The use of a stairway for entry is dictated by the requirement for short access time; the length of the legs is governed by the radiation criteria; the location of the generator and collective-protector compartments is established by the need for access and for isolation of these items of equipment. Ingress through either the primary or the alternate entranceway can be accomplished at a rate of about 60 persons per minute, permitting the shelter to be filled in about 2 minutes. Entry of contaminated persons would be much slower because of the need for showers.

Blast protection for additional persons is provided by the leg of the "T" in the primary entranceway. Figure 4 indicates the point at which prompt radiation reaches an unsafe level.

The design for the primary entranceway is detailed in Appendix C.

An alternate entranceway (Drawing No. 936971) is proposed when all-concrete construction is desired. The radiation design for the alternate entranceway follows the same procedure as that used for the primary entranceway. The same type of blast door is proposed for both. The structural design of the alternate entranceway is straightforward, using principles set forth in the literature.[30]

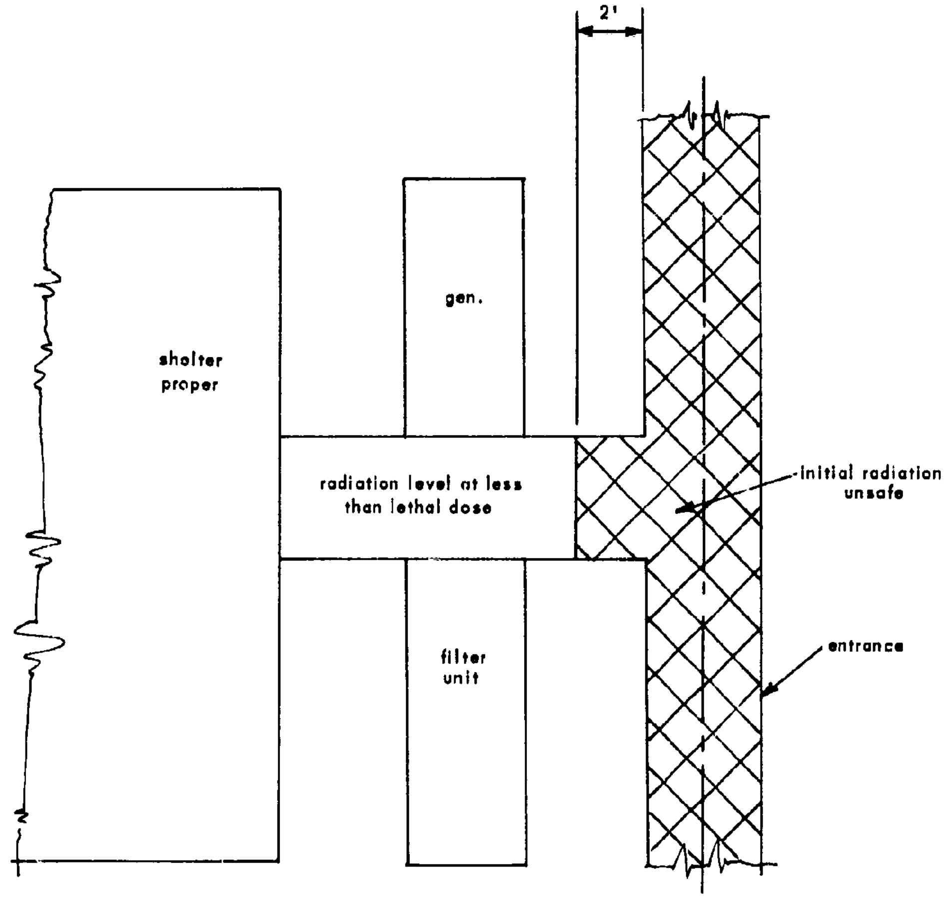


Figure 4. Danger limit for initial radiation.

In addition to providing radiation protection, the entry must be designed to withstand the air-blast-induced ground shock and the differential motions which will occur. To accommodate the differential movements, a clearance of 2 inches is allowed between the tube and the shelter and between the tube and the blast wall. No difficulty from differential movements is expected in the metal-to-metal connections.

Blast Door. A blast door for an operational shelter must have four primary characteristics. It must (1) resist the design blast pressure, (2) be capable of being opened and closed quickly, (3) be readily available, not overly expensive, and have sufficient width to permit rapid entry of personnel and handling equipment. These characteristics are admirably met in the tension-type door developed by the Germans, hence, it is employed. The particular door specified for the shelter, a 32-inch by 68-inch medium steel door, is depicted in Appendix E.

A concrete blast wall is required to anchor the blast door and to transfer the load on the door to the soil. Details of the blast wall are shown in Drawing No. 936962. The concrete is feathered to the outside tube so that the blast wave will not rip out the wall.

Secondary Doors. The doors at the entrance to the shelter proper, to the generator space, and to the collective-protector room must be airtight. Navy quick-acting doors are specified, inasmuch as they quite likely will be available from Navy salvage yards. In addition to their quick-opening characteristics, these doors have the advantage of providing secondary blast protection.

Other doors in the shelter serve no critical function other than to divide spaces; they are described in the drawings and specifications.

Stairs and Floor. The steps in the entrance are of expanded metal. The attachments are of sufficient strength to resist damage from the shock wave provided they are not struck by missiles. A concrete floor is used in other portions of the entry as shown in the drawings.

Water Seals. Water seals are afforded at the juncture of the inner tube with the shelter and the blast wall by gluing a plastic-strip seal to the adjacent components. Such a strip will seal the 2-inch gap and allow for some differential motions between components.

Drainage. Both legs of the entranceway are drained to a sump located outside the blast door. The sump is covered with a grill. Drainage from the sump to an exterior drainage line must be provided.

Footing Design

The hypothesis has been posed that it is desirable to have the foundation act independently of the floor slab and for the footing to punch into the supporting soil when it is subjected to blast loading. The amount of punching, of course, must be limited to a few inches. The validity of the hypothesis is subject to question on these grounds: (1) multiple loading could result in excessive deflections, and (2) if the load carried by arching across shallow-buried structures is small (as indicated by recent unpublished tests on model arches) there is little to be gained by permitting the foundation to punch into the soil. Since better information is not available, the foundation for the recommended shotcrete shelter is based on the afore-stated hypothesis.

The footings are designed to limit the deflection of the footing relative to the floor slab to 3 inches. It is deflection and not bearing capacity which determines the size of footing required. Unfortunately, precise calculation of deflection is not possible with present knowledge because of the lack of dynamic soils test data. Not only is data lacking on the dynamic properties of soil but information on the static properties is incomplete. For example, there is little information on the variation of bearing pressure or passive pressure with depth or with shape and size of footing. Available information is usually from tests of scaled-down experiments[35] wherein the applicability of results to full-size structures remains subject to question. The design method developed and employed here is dependent upon the cited limitations in knowledge.

The approach used in the design of the footings, as developed in Appendix D, is as follows:

1. The required width of footing is estimated based on the static loading.

2. The entire arch is treated as a beam on an elastic foundation to determine the flexural steel required in the footing, the diagonal tension steel required in the arch, and the deflection at mid-span caused by flexure.

3. The need for tying the foundations together is explored.

4. The required reinforcement to resist footing torsion is considered.

5. The permanent deflection due to punching of the footings into the soil is estimated by use of the single-degree-of-freedom analogy.

6. The design is revised as necessary.

The footings, as defined in Drawing No. 936963, are based upon the specific soil conditions given on page 68. Other soil characteristics may require design modifications.

Other Design Considerations

Most features of the shelter other than those considered above are amenable to design by usual methods and are, therefore, not treated here. Several facets of the design are worthy of mention or explanation; namely, the interior partitions, the seal between the floor and the foundation, the intake and exhaust termini, the emergency blower, the means of maintaining pressure in the shelter, and the costs.

The interior partitions must transmit the load of the water tanks through the floor to the soil; under ground-shock conditions, the load can be sizable. The best information available indicates that the tanks would be subjected to a maximum acceleration of 8 g's. Based on this acceleration, the partitions and floor are quite adequate to carry the induced loading provided a modest amount of reinforcement is added to the floor slab in the vicinity of the partitions as shown in Drawing No. 936962. Obviously, the tanks must be anchored against moving on their mounts.

The seal between the floor and the foundation is designed to maintain its integrity even after large relative displacement. The seal is not expected to prevent leakage under high pressures, but this should not be a problem except in areas where the water table is high. Sealing is one of the many aspects of shelter design which warrant in-service testing. A membrane waterproof cover is used on the exterior of the shelter.

Another component which warrants in-service testing is the external intake and exhaust termini shown in Drawing No. 936967. The ventilation lines exterior to the shelter terminate with a "T" embedded in crushed rock below the ground surface, thus precluding the possibility of damage from the dynamic pressure and the debris which it carries. Debris and dust might clog the intake rock filter, but in such an emergency it could probably be cleaned out quickly by one of the shelter occupants.

Another emergency measure would be "cutting in" the hand blower in the event of failure of the filter unit's blower or the generator. This would be accomplished by closing the valve to the regular air-distribution duct and opening the valve in the hand-blower line.

During occupancy of the shelter, it is important that a pressure of about 0.5 inch of water above ambient pressure be maintained in the shelter. This is oftimes difficult to accomplish unless particular care is taken in sealing the structure. The interior pressure is stepped down to 0.3 inch of water in the shower and finally to 0.1 inch of water in the inner entry by means of air pressure regulators in the doors to the shower. A manometer should be installed in the structure to enable the occupants to determine the shelter pressurization at any given time.

Costs

Direct costs for the shelter as designed are listed in Table III. The total direct cost for labor and materials is $43,043; allowing 28 percent for overhead, liability and insurance, social security and unemployment taxes, profit, and bonds, the estimated cost for construction of the shelter is $55,000. The estimates are based upon current State of California labor rates and materials prices in the Los Angeles area. For the State of New York the estimates would be increased by 14 percent.

Use of a salvageable Quonset instead of a new one would decrease the form costs to $3,100. A plywood form for conventionally placed concrete costs $6,020; hence a considerable saving can be realized by the use of shotcrete. Also, in construction of two or more shelters, the form could be reused a number of times, thereby considerably reducing the form cost per shelter.

From the last column of Table III, which expresses the cost of each item as a percent of the total direct cost, it is seen that the entranceway cost is the largest single item. It amounts to 18.2 percent of the total direct cost. The next three larger are the costs of the forms, backfill, and electrical work. Each of these is approximately 11.5 percent of the total. Attempts at reducing the cost of the shelter should be directed at these items.

In situations where only able-bodied persons are to be using a shelter it may be possible to use a chute, a vertical slide pole, or a vertical ladder to cut the cost of the entrance. Fortunately, several new concepts for blast-closure valves are currently under development which are certain to markedly reduce the cost indicated in Table III. Reducing the costs through multiple use of the forms, use of simple entranceways where possible, and reduction in the cost of blast-closure valves should enable the total direct cost to be reduced to within $35,000.

Table III. Shelter Costs

Item	Direct Cost* (dollars)	Percent of Total Cost
Excavation	2,700	6.3
Backfill	4,950	11.5
Shotcrete	2,800	6.5
Reinforcing steel	3,072	7.1
Forms	5,000	11.6
Misc. carpentry	563	1.3
Misc. iron and sheetmetal work	4,538	10.5
Entranceway, complete with stairs, sump, doors, etc.	7,828	18.2
Water seals	391	0.9
Electrical work	4,837	11.3
Blast-closure valves	3,500	8.1
Filter unit	1,334	3.1
5-kw generator	1,530	3.6
Total	43,043	100%

* Cost of labor, materials, and equipment charges

DISCUSSION

Comparisons between shelters are desirable but difficult to effect. For example, because of differences in design criteria, it is essentially impossible to make a meaningful comparison of the so-called gable shelter [36] and the concrete shelter presented here. The gable shelter was designed in the spring of 1953 prior to Operation Upshot/Knothole — before the detonation of the first megaton weapon in November of 1952. Since that time the entire protective-construction philosophy and approach has changed. It is felt, therefore, that any attempt to effect a comparison between the gable shelter and the shotcrete shelter would be meaningless.

A second comparison, which suggests itself to those familiar with the protective-construction field, is between the Navy standard corrugated-metal shelter and the shotcrete shelter. Here again a direct comparison is impossible because the standard metal shelter with ribs is only rated at 75 psi capacity. (The concrete shelter defined here is designed for 100 pounds per square inch overpressure.) Functional arrangements could, of course, be compared with modest gain, but this would be removed from the prime purpose of this report.

Considerable additional detailed information on functional arrangement, supplies, and equipment is available elsewhere,[7,55] and therefore is not repeated here. Additional information may also be gleaned from shelters which have been recently designed and built. Among these are the Navy's shelter at the Naval Medical Center, Bethesda, Maryland, Y&D Drawings No. 881040 through 881047. It had a circulating water-supply system which could be employed with the concrete personnel shelter. A similar shelter designed primarily for fallout protection is in existence at Camp Parks, California, and has been used for several habitability studies. A modification of this shelter, with a rearranged entry, has been built at the Construction Battalion Center, Port Hueneme, California, and is scheduled for use in control and operational studies. Another corrugated-metal shelter with many interesting mechanical details was designed for Operation Trumpet, Y&D Drawings No. 813481 through 813493, but has not been built. Other shelters of the type being discussed have been designed, but as yet few shelters have been built and even fewer have been subjected to in-service testing.

Thus, the adequacy of contemporary design methods remains subject to opinion. This is well illustrated by comparing the shell thicknesses in Table IV required by some of the more credible methods. Methods A,[37] B,[38] and C[39] are equivalent static-load methods whose loading is based upon the judgment of the respective authors. These judgments give results which vary by a factor of almost two. Which one is correct, if any, will remain unknown until further test data become available.

Table IV. Comparison of Shell Thickness From Three Design Methods*

Method	Shell Thickness (in.)	Reinforcement (%)
A	4.5	0.5
B	8.0	0.5
C	6.5	0.5

* All calculations were based on an ideal blast wave with a peak overpressure of 100 psi and a duration of 250 milliseconds. Working stresses were as follows:

Reinforcement f_s = 40,000 psi; 25 percent increase in yield point for dynamic loading.

Concrete f'_c = 5,000 psi; E = 4,100,000 psi.

Arch radius r = 10 feet.

As do many studies, this one has served to emphasize unknowns and generate questions. The unknowns are primarily those of the basic static and dynamic properties of soils and of the mechanics of soil-structure interaction. Some of the questions are as follows:

1. Is 100 psi the optimum design overpressure level? Recent studies[19] indicate that 250 psi is a more nearly optimum value for civil structures.

2. Is the earth cover and the thickness of the concrete shell sufficient to provide protection from large high-velocity fragments and other missiles?

3. Should shelters be built below the ground-water table in areas where the water table is high; viz., around most Navy bases?

4. Should cylindrical closed shelters be used in high-water-table regions to obviate the sealing problem inherent in arch structures?

5. What is the difference in the response of buried structures to long- and short-duration loads?

Innumerable lesser problems exist, but these questions serve to emphasize the interim nature of the solution represented by the drawings of Appendix E. Considerable improvement in the design should be possible in the near future as the results of the rather intense research efforts presently underway become available. The plans are looked upon as a conservative nucleus to which refinements can be made in seeking the optimum shelter design.

ACKNOWLEDGMENTS

The authors extend their appreciation to Messrs. W. A. Keenan and G. E. Sherwood for their aid in establishing the architectural layout, and to Messrs. S. L. Bugg and R. A. Breckenridge who assisted in formulating the design criteria. Credit is also due to members of the Laboratory staff who served as consultants on aspects of the shelter design within their fields of specialty and to those persons in the Bureau of Yards and Docks who reviewed the report and offered many helpful suggestions.

REFERENCES

1. Department of the Navy, Bureau of Yards and Docks. "Task Instruction for FY 62 NCEL Program," 30 August 1961.

2. Department of the Navy, Bureau of Yards and Docks. P-81, "Personnel Shelters and Protective Construction," 1962.

3. U. S. Naval Civil Engineering Laboratory. Technical Report R-123, "Methods of Shotcrete Construction for Personnel Shelters," by R. M. Webb. Port Hueneme, California, 17 March 1961.

4. Operation Plumbbob, Project 3.1, WT-1420, "Blast Loading and Response of Underground Concrete-Arch Protective Structures," by R. A. Breckenridge, W. M. Flathau, and C. K. Wiehle, 5 June 1959, (Confidential).

5. Operation Plumbbob, Project 3.3, ITR-1422, "Evaluation of Earth-Covered Prefabricated Ammunition Storage Magazines as Personnel Shelters," by G. H. Albright, 1959.

6. Operation Hardtack, Project 3.2, ITR-1626-1, "Response of Earth-Confined Flexible-Arch-Shell Structures in High-Overpressure Region," 5 September 1958, (Secret); and Supplement, ITR-1626-2, by G. H. Albright, P. J. Rush, J. C. LeDoux, V. True, and C. R. White, 10 April 1959, (Secret).

7. Naval Radiological Defense Laboratory. Technical Report R-366, "Specifications and Costs of a Standardized Series of Fallout Shelters," by W. E. Strope, L. G. Porteous, and A. L. Greig. San Francisco, California, 6 October 1959.

8. Department of the Navy, Bureau of Yards and Docks. TP-PL-2, "ABC Warfare Defense Ashore," April 1960.

9. Swalley, R. F., and J. R. Allgood. "Specifications for Military Personnel Shelters," The Navy Civil Engineer, March 1961.

10. West German Federal Civil Defense Agency. "Report on the Preparations for an Execution of an Air Raid Shelter Manning Test in 'Shelter' of the Waldrol Federal Air Raid Protection School." West Germany, January 1959.

11. Naval Radiological Defense Laboratory. Technical Report R-418, "Preliminary Report on the Shelter Occupancy Test of 3-17 December 1959," by W. E. Strope. San Francisco, California, 4 May 1960.

12. U. S. Army Engineer Research and Development Laboratories. Project 2230.1, "Hasty Personnel Shelter for Protection from Atomic Effects," by R. G. Marshall and J. E. Pleasants. Fort Belvoir, Virginia, 15 November 1958.

13. Office of Civil and Defense Mobilization. "Inclusion of Fallout Shelters in Buildings," Advisory Bulletin No. 243, 24 August 1959.

14. Panero, Robert. "Minimum Design Standards for Radiation Fallout Shelters," Air Conditioning, Heating and Ventilating Magazine, October 1959.

15. U. S. Army Engineer Waterways Experiment Station, Corps of Engineers. "Analysis and Design of Flexible Underground Structures," by N. M. Newmark, J. W. Briscoe, and J. C. Merritt. Vicksburg, Mississippi, 1 May 1960.

16. U. S. Atomic Energy Commission. Civil Effects Test Operation, "AEC Group Shelter," (from CEX-58.7), Holmes and Narver, Inc., 22 June 1960.

17. "Shelter Designs for Protection Against Radioactive Fallout," by Voorhees, Walker, Smith, Smith, and Haines, New York, 1 February 1960.

18. American Institute for Research. "Psychological and Social Adjustment in a Simulated Shelter," by J. W. Altman, et al. Prepared under Contract ODM-SR-60-10 sponsored by OCDM. Pittsburg, Pennsylvania, November 1960. Appendixes issued under separate cover.

19. Office of Civilian Defense and Mobilization, Operations Research Office. SA-61-12, "Shelter," by J. F. Devaney, 1961.

20. U. S. Atomic Energy Commission. "The Effects of Nuclear Weapons," Edited by Samuel Glasstone. Washington, D. C., June 1957.

21. U. S. Naval Civil Engineering Laboratory. Technical Report R-025, "Nuclear Radiation Shielding Provided by Buried Shelters," by J. C. LeDoux. Port Hueneme, California, 27 October 1959.

22. Armour Research Foundation of Illinois Institute of Technology. ARF, No. K 158, Final Report, "Design of Entrance Systems for Personnel Protective Shelters," by R. L. Barnett, (Secret). (Chapter 7, "Movement of Personnel," Unclassified.) Chicago, Illinois, December 1959.

23. U. S. Naval Civil Engineering Laboratory. Technical Note N-353, "Shelter Habitability Studies — the Effect of Odor in a Shelter, and Ventilation Requirements," by J. S. Muraoka. Port Hueneme, California, 12 October 1960.

24. Department of the Army, Corps of Engineers. EM-1110-345-461, "Manual for Engineering Design. Collective Protection Against Chemical, Biological, and Radiological Warfare Agents."

25. Heating, Ventilating and Air Conditioning Guide. ASHVE, 1958.

26. Office of Naval Research. NRL R-5465, "The Present Status of Chemical Research in Atmospheric Purification and Control on Nuclear-Powered Submarines," by R. R. Miller and V. R. Piatt. Washington, D. C., 21 April 1960.

27. U. S. Army, Chemical Research and Development Laboratories. CRDLR 3067, "The Effect Upon Albino Rats of Chronic Exposure to Various Concentrations of CO_2," by P. F. Robinson. Army Chemical Center, Maryland, May 1961.

28. Ballistic Research Laboratories. BPTC-60, DASA 1171, "Blast Patterns in Tunnels and Chambers; First Information Summary." March 1960.

29. U. S. Army, Corps of Engineers. "Design and Analysis of Underground Reinforced-Concrete Arches," by W. J. Flathau, et al. Vicksburg, Mississippi, March 1961, (Draft).

30. Anderson, F. E., Jr., et al. "Design of Structures to Resist Nuclear Weapons Effects," ASCE Manual No. 42, 1961.

31. Selig, E. T., K. E. McKee, and E. Vey. "Underground Structures Subject to Air Overpressure," J. of Engineering Mechanics Division, ASCE, August 1960.

32. Hognestad, Eivind. "Yield-Line Theory for the Ultimate Flexural Strength of Reinforced Concrete Slabs," J. of ACI, Vol. 24, No. 7, March 1953.

33. U. S. Naval Civil Engineering Laboratory. Technical Report R-121, "Design Charts for R/C Beams Subjected to Blast Loads," by J. R. Allgood, and G. R. Swihart. Port Hueneme, California, 9 January 1961.

34. Spangler, M. G. Soil Engineering. International Textbook Company, Scranton, Pennsylvania, 1951.

35. Air Force Special Weapons Center. Technical Report 59-56, "Design and Analysis of Foundations for Protective Structures," by K. E. McKee. Albuquerque, New Mexico, October, 1959.

36. Department of the Navy, Bureau of Yards and Docks. TP-PL-8, "Personnel Shelters and Protective Construction." BuDocks 11010, 25 November 1958, including Appendix A.

37. U. S. Army Corps of Engineers. Manual, EM 1110-345-420, "Design of Structures to Resist the Effects of Atomic Weapons; Arches and Domes," 15 January 1960.

38. Newmark, N. M. "Vulnerability of Arches." Presented at the ASCE Meeting in Reno, Nevada, May 1956.

39. "Analysis of Operation Plumbbob, Project 3.1 Structures," by Holmes and Narver. Work performed under Contract DA-22-079-ENG-196, Modification No. 3, for the Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi, April 1958.

40. Hodge, P. G., Jr. Plastic Analysis of Structures. McGraw-Hill, New York, 1959.

41. Allgood, J. R., and G. R. Swihart. "Design of Flexural Members for Static and Blast Loading." Draft of ACI Monograph, March 1961.

42. "The ACI Building Code," (ACI 318-56), J. of ACI, September 1947, Proceedings, Vol. 44, February 21, 1956.

43. LeDoux, J. C., and A. B. Chilton. "Transmission of Gamma Rays Through Two-Legged Rectangular Ducts," ANS Paper No. 4, Session No. 4, Winter, 1960.

44. Bureau of Yards and Docks. Technical Study No. 27, "Air Blast Induced Ground Shock," by W. J. Christensen, September 1959.

45. Operation Plumbbob, Project 3.2, WT-1421, "Evaluation of Buried Conduits as Personnel Shelters," by R. A. Breckenridge, 1960.

46. California Institute of Technology. "Vibration Compaction of Sand." Pasadena, California, January 1952 (submitted in partial fulfillment of a contract sponsored by NCEL).

47. Hetenyi, M. Beams on Elastic Foundations, 1st Edition. University of Michigan Press, Ann Arbor, 1946, p. 82.

48. Gibson, J. E., and D. W. Cooper. The Design of Cylindrical Shell Roofs, 1st Edition. D. Van Nostrand, Princeton, New Jersey, 1954, pp. 37-49.

49. Ernst, G. C., R. R. Marlette, and G. V. Berg. "Ultimate Load Theory and Tests of Cylindrical Long Shell Roofs," J. of ACI, Vol. 26, No. 3, November 1954.

50. Ernst, G. C. "Ultimate Torsional Properties of Rectangular Reinforced Concrete Beams," J. of ACI, Vol. 29, No. 4, October 1957.

51. Timoshenko, S. Strength of Materials, Part II, 2nd Edition. D. Van Nostrand, New York, 1941, p. 266.

52. Norris, C. H., et al. Structural Design for Dynamic Loads. McGraw-Hill, New York, 1959.

53. Cohen, E., and S. Weissman. "Underground Shock Environment Data and Application to the Design of Underground Structures," Shock, Vibrations and Associated Environments, Part III, Bulletin No. 28, September 1960. Office of the Secretary of Defense, Research and Engineering, Washington, D. C.

54. Zinkahn, W., and H. Leutz. "Comments on Protective Construction in Germany," ONI Translation No. 1338, 1956.

55. Department of the Navy, Bureau of Yards and Docks. Publications P-290, P-290.1, P-290.2, "Studies in Atomic Defense Engineering," January 1957, August 1958, November 1961.

LIST OF SYMBOLS

A	=	constant of integration
A_n	=	radiation attenuation for leg, n = 1, 2, 3...
A_s	=	area of steel
A_{s_b}	=	area of steel in bottom of footing to resist F_{t_b}
A_{s_t}	=	area of steel in top of footing to resist F_{t_t}
A_v	=	area of stirrup
a	=	length of footing; cross-sectional dimension for torsion analysis
B	=	constant of integration
b	=	width of footing; linear dimension
b_e	=	width of rectangular section, inches
b'	=	reinforcing cage width of equivalent section
C	=	constant of integration; detrusion coefficient; linear distance
C_{k_z}	=	spring-constant coefficient
c	=	slope ratio; distance to extreme fibre; linear distance; coefficient of viscous damping
c_o	=	constant
c_2	=	distance from centroid of tension steel to tension face of concrete
D	=	diameter; depth below ground of contact face of footings
D_{ew}	=	thickness of endwall
D_f	=	elevation of footing in calculations for ultimate bearing capacity
D_{s2}	=	depth of two-way slab for minimum cost

d = depth of member; depth from compression face to centroid of tensile steel

d' = depth from compression face to centroid of compression steel

E = modulus of elasticity of soil

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of steel

E' = increase in E per unit depth

e = displacement

F = D'Alembert's inertia force

F_n = loads on arch, n = 1, 2, 3...

F_{t_b} = total tensile force in an incremental depth of footing measured from bottom of footing

F_{t_t} = total tensile force in an incremental depth measured from top of footing

f_c = compressive force

f'_c = compressive strength of concrete

f_s = stress in tensile steel

f_t = tensile stress

$f(t)$ = forcing function

f_v = stress in shear steel

f_y = yield point of reinforcement

G = a factor which accounts for the radiation from prime scattering areas

H = depth of footing

H_a = horizontal reaction

H_z = depth of soil media

h = equivalent depth of soil

I = moment of inertia

I_{B_o} = "I" of footing about its top

I_n = "I" of the beam

I_{s_o} = "I" of arch about its base

j = ratio of distance between centroid of compression and centroid of tension to depth d

K_p = modulus of passive pressure

K_z = spring constant of analogous system

k = foundation modulus in units of pounds per cubic inch; kip; ratio of lateral pressure to ground-surface overpressure

k_b = foundation modulus (spring constant for the beam) in units of pounds per inch squared

k_o = foundation modulus in units of pounds per cubic inch obtained from plate-bearing tests; initial tangent to load-deflection curve

k_p = spring constant in units of pounds per inch of deflection obtained from plate-bearing test

k_z = foundation modulus in units of pounds per inch

L = length of the beam

L_n = length, n = 1, 2, 3...

M = moment

M_c = moment at the center of the beam

M_p = plastic moment

M_μ = ultimate moment

m = mass; unit moment

N = resultant of soil-pressure forces

N_i = correction factors in ultimate bearing capacity equations

N_q = correction factors in ultimate bearing capacity equations

N_x = maximum compressive force within the hypothetical beam

n = distance from neutral axis to extreme fibre; a constant determined from experiment

P = total load

P_n = upward load on end of beam

p = total pressure on a differential strip of soil; pressure on arch at any time

p_a = lateral hydrostatic pressure

p_f = foundation pressure beneath the footing

p_o = peak overpressure

p_p = passive lateral pressure

q = unit pressure beneath plate or footing at limit of region of elastic behavior, $lb/in.^2$

q_b = uniformly disturbed load, lb/in.

q_d = ultimate shear strength of the soil

R = maximum resistance

r = radius; effective radius; length-to-width ratio

S = axial thrust

S_f = frictional forces on the sides of footings

s = footing displacement parameter; shear steel spacing

T = maximum thrust; period of vibration; transmission factor; torque

T_e = torque in the footing

T_t = total reaction of endwall footing

t = thickness of the arch

t_e = depth of equivalent section

t_1 = effective duration of the blast, seconds

t' = reinforcing cage depth of equivalent section

u = equivalent hydrostatic uplift pressure

V = total shear

$V_{(\,)}$ = vertical reaction at point subscripted

v = unit shear

v_y = ultimate unit shear stress

W = yield of the weapon, megatons

w = load per unit area on endwall

w_{bm} = uniformly distributed load on a simple beam

w_n = width of tube, n = 1, 2, 3...

y = displacement of spring-mass system

y_c = deflection at center of beam

y_e = yield resistance

z = variable depth within the soil foundation

$\bar{z}$ = location of neutral axis within the cross section of the arch

α = constant in torsion equation

β = loading coefficient

$\beta_{n.a.}$ = angle giving location of the neutral axis of the arch cross section; a constant determined from experiment

γ = displacement

Δ_w = change in work

δ = unit displacement

δ_o = total displacement within a soil mass

ϵ = unit strain

θ = angle of twist; angular coordinate; angular rotation

θ_e = rotation at the support of the hinged-end arch

λ = constant in torsion equation; characteristic length of the hypothetical beam

ρ = density

φ = percent of reinforcement; angle of internal friction

ψ = angular rotation

ω = frequency

$\$_c$ = cost of concrete in place

$\$_s$ = cost of reinforcement in place

Appendix A

ARCH DESIGN

The buried arch shown schematically in Figure A1 is assumed to be loaded as shown in Figure A2. This load distribution is justified on the basis that it approximates the load distribution at maximum deflection of structures tested at the Nevada Test Site[4] and small-size structures tested in the Laboratory's blast simulator.

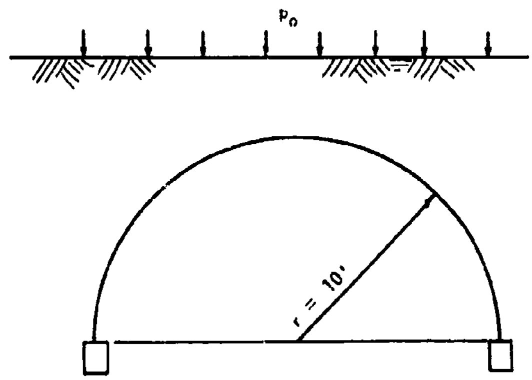


Figure A1. Buried arch.

With the loading of Figure A2 an ultimate design procedure is employed using the assumed mechanism of Figure A3. The Upper Bound Theorem[40] assures that a load computed on the basis of an assumed mechanism will always be greater than or equal to the actual ultimate load.[37] F_1 and F_3 in Figure A3 represent loads on the mechanism which are equivalent to the assumed load on the structures shown in Figure A2. F_2 is the force which represents the resistance due to passive pressure in the soil.

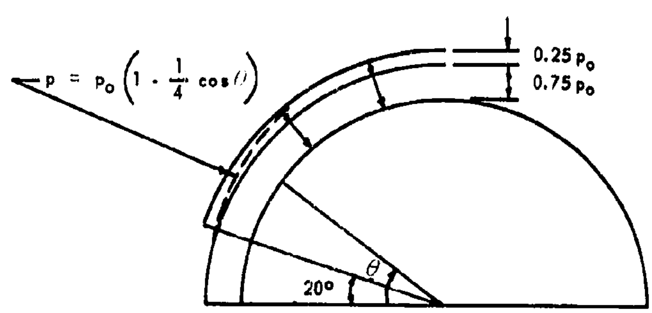


Figure A2. Loading on arch.

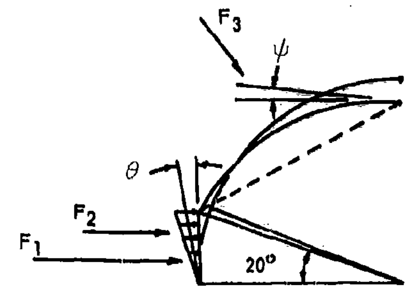


Figure A3. Assumed mechanism and equivalent loading.

The procedure for developing the design is to calculate the equivalent loads, establish the geometry relations, write a virtual-work equation, calculate the depth for minimum cost, and solve for the area of steel required.[30]

For a 1-foot width of arch, the loads are

$$F_1 = 2 \times \frac{3}{4} p_o r \sin 10^\circ = 1.5 \times 100 \times 144 \times 10 \times \sin 10^\circ \tag{A1}$$

$$= 37{,}500 \text{ lb}$$

$$F_3 = 2 p_o r \sin 35^\circ = 2 \times 100 \times 144 \times 10 \times \sin 35^\circ \tag{A2}$$

$$= 165{,}000 \text{ lb}$$

$$F_2 = \frac{K_p}{2} (2 r \sin 10^\circ)^2 \theta \tag{A3}$$

For $K_p = 174 \text{ lb/in.}^3$:

$$F_2 = \frac{174 \times 1{,}728}{2} (2 \times 10 \times \sin 10^\circ)^2 \theta$$

$$F_2 = 1.81 \times 10^6 \times \theta$$

The geometry relations required for the virtual-work equation, including an expression for the angle, φ, in terms of θ, are written with reference to Figure A4:

$$c = \sqrt{L_2^2 - b^2}$$

and

$$\gamma = r(1 - \sin 20^\circ) - c$$

$$L_2 = 2 \times 10 \times \sin 35^\circ = 11.48'$$

$$b = 2 \times 10 \times \theta \sin 10^\circ + 10 \sin 70^\circ = 3.47\theta + 9.40$$

$$c = \sqrt{11.48^2 - (3.47\theta + 9.40)^2} = \sqrt{-12.04\theta^2 - 65.2\theta + 43.43}$$

and

$$\gamma = 10(1 - \sin 20^\circ) - c = 6.58 - c$$

The distance through which F_3 moves is

$$e \cong \frac{L_2}{2} \varphi = 5.74\varphi$$

where

$$\varphi = \tan^{-1} \frac{r(1 - \sin 20^\circ)}{r \sin 70^\circ} - \tan^{-1} \frac{c}{b}$$

$$\varphi = \tan^{-1} \frac{6.58}{9.40} - \tan^{-1} \frac{\sqrt{-12.04\varphi - 65.2\varphi + 43.4}}{3.47\varphi + 9.40} \qquad \text{(A4)}$$

The virtual-work equation is

$$\Delta_w = 0 = F_3 e - F_1 \theta r \sin 10^\circ - F_2 \frac{2}{3} \times 2r(\sin 10^\circ)\theta$$

$$- M_p(\theta + \varphi) - M_p \varphi \tag{A5}$$

or

$$165{,}000 \times 5.74\varphi - 37{,}500 \times 10(\sin 10^\circ)\theta$$

$$- 1.81 \times 10^6 \theta^2 \frac{4}{3} \times 10 \sin 10^\circ - M_p\theta - 2M_p\varphi = 0 \tag{A6}$$

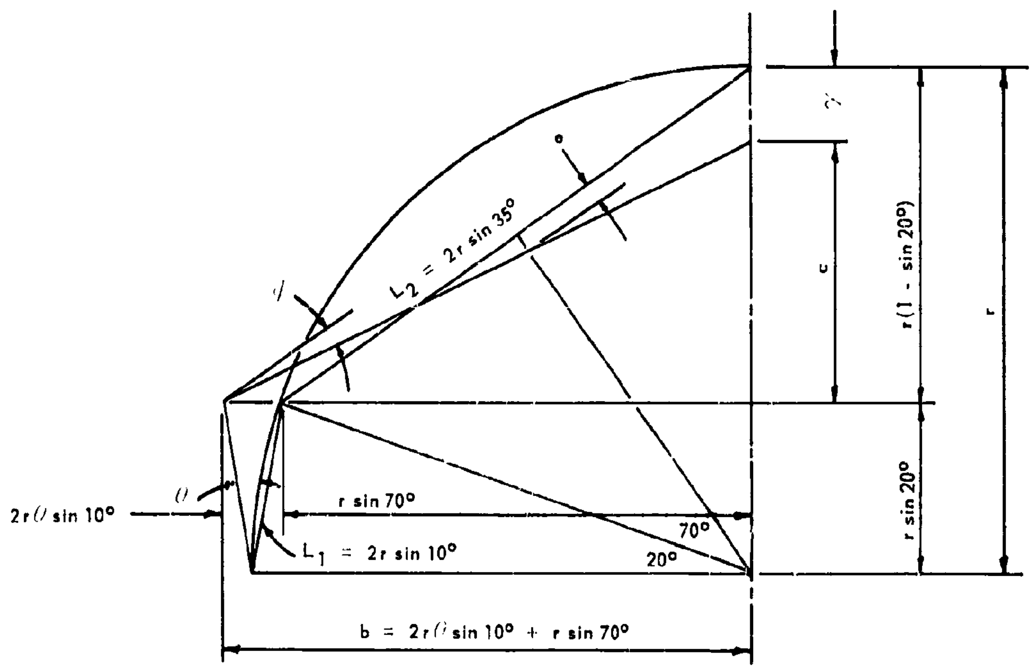


Figure A4. Geometry of mechanism.

From this relation it may be seen that the required section is a function of the tolerable hinge rotation. A study of the ratio φ/θ with changes in θ shows that the ratio is approximately constant and, for practical purposes, may be taken equal to its value at θ equals 5 degrees, which is 0.54. With this ratio, the last equation becomes

$$165{,}000 \times 5.74 \times 0.54 - 37{,}500 \times 10 \times \sin 10^{\circ}$$

$$- 1.81 \times 10^{6} \times \frac{4}{3} \times 10(\sin 10^{\circ})\theta = 2.08M_p$$

or

$$2.08M_p = 511{,}000 - 65{,}000 - 4.19 \times 10^{6}\theta \qquad \text{(A7)}$$

For small values of θ the last term is negligible, and

$$M_p = \frac{446{,}000}{208} = 214{,}000 \text{ ft-lb/ft}$$

Next, the depth for minimum cost may be calculated, depending on local costs for concrete and reinforcement;[41] or a depth may be assumed. Using a depth of 10 inches and a 25-percent increase in yield point because of the dynamic loading, the required area of steel is

$$A_s = \frac{M_p}{1.25 f_y jd} = \frac{214{,}000}{1.25 \times 40 \times 10^{3} \times .875 \times 7.5} = 0.652 \text{ in.}^2/\text{ft}$$

Use #5 at 6 inches = 0.62 in.^{2}/ft

In selecting the section, the last term in Equation A7, which represents the resistance from passive pressure in the soil, was neglected. Yet, the magnitude of this term becomes very large for large values of θ. The reason for neglecting the passive pressure term was to limit the extent of cracking in the structure at the design load. The actual ultimate load capacity, however, will be considerably in excess of the design load.

The cited conservatism is desirable because of: (1) the possibility of multiple loading, (2) the added resistance against major cracking and subsequent water intrusion, and (3) the cost to gain the added resistance of the arch is small.

It is interesting to note that the resistance developed from passive pressure is primarily responsible for preventing large lateral motions and for maintaining an essentially radial loading on the arch. Thus, a properly backfilled buried arch of modest proportions becomes a veritable fortress.

Appendix B

DESIGN OF ENDWALLS

Design of the endwalls by the yield-line theory has the advantage of providing results quickly and easily. Such a solution is superior to the approximate methods recommended by the ACI [42] because the resulting yield-line patterns aid the designer in proper layout of the reinforcement. Further, an "equivalent simple beam" may be defined by using the moment-load relation from the yield-line theory which permits a dynamic solution by use of available charts. [33]

As an example, the design of the endwall without the opening for the door is considered. Design of the endwall is based on the assumption that the edges of the endwall are fixed and that the compression load "dumped" into the endwalls from the arch has a negligible effect on the lateral load capacity of the endwall. By assuming a yield-line pattern as shown in Figure B1, the moment per unit length, m, for the various parts is found in terms of the applied lateral load, w, as follows:

Part	(1) Area	(2) Dist. to Centroid	(1) x (2)	Base
A	58.0	1.99	115.4	20.0
B	24.0	1.75	42.0	9.2
C	17.7	1.9	33.6	6.2

Part A

$$20 \times 2m = 115.4w$$
$$m = 2.89w$$

Part B

$$9.2 \times 2m = 42.0w$$
$$m = 2.28w$$

Part C

$$6.2 \times 2m = 33.6w$$
$$m = 2.71w$$

$$m_{avg} = 2.62w$$

By the method of virtual work:

$$m\left(\frac{2 \times 20}{5} + \frac{2 \times 2 \times 9.2}{5.2} + \frac{2 \times 2 \times 6.2}{4.3}\right) = w\left(\frac{115.4}{5} + \frac{2 \times 42}{5.2} + \frac{2 \times 33.6}{4.3}\right)$$

from which $$m = 2.67\,w$$

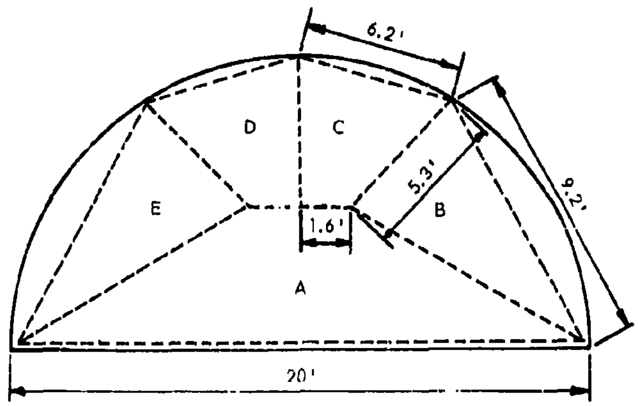


Figure B1. Yield-line pattern.

If the correct yield-line pattern had been assumed the moment-load ratio would be the same for each part of the endwall. Adjusting the yield lines by trial or employing the method of virtual work, the corrected moment-load ratio is found to be 2.67.

Thus, $$m = 2.67\,w$$

or, $$w = 0.374\,m$$

For a simply supported beam with a yield moment, m, and a span of 10 feet,

$$w_{bm} = \frac{8m}{L^2} = 0.08m$$

Therefore, $$\frac{w_{bm}}{w} = \frac{0.08m}{0.374m} = 0.214$$

Assuming that the endwall receives 0.5 of the overpressure and that the load factor is 1.5, the load on the "equivalent simple beam" is

$$w_{bm} = \frac{0.214 \times 100 \times 144 \times 1.5}{2} = 2,310 \text{ psf}$$

Next, the thickness of endwall for minimum cost[41] is calculated (or a thickness may be selected based upon the judgment of the designer). The depth for minimum cost may be found from the relation,

$$D_{s2} = c_o + 2\sqrt{\frac{M_\mu \$_s}{f_y b \$_c}}$$

$$c_o = d' + c_2 = 2.5 + 2.0 = 4.5$$

$$M_\mu = \frac{wL^2}{8} = \frac{2,310 \times 10^2}{8} = 28,900 \text{ ft-lb/ft}$$

$$\$_s = 0.050 \text{ \$/in.}^3$$

$$\$_c = 0.00059 \text{ \$/in.}^3$$

$$f_y = 40,000 \text{ psi}$$

$$b = 12 \text{ in.}$$

$$D_{s2} = 4.5 + 2\sqrt{\frac{28,900 \times 12 \times 0.050}{40,000 \times 12 \times 0.00059}} = 20.1 \text{ in.}$$

Since the average span used to compute the moment is less than 10 feet, this thickness is considered to be excessive; use D_{s2} = 18 inches.

Thus,

$$\frac{L}{d} = \frac{10 \times 12}{15.5} \cong 7$$

Selecting $d'/d = 0.15$, assuming $t'/T = \infty$, and extrapolating slightly in Figure 15 of TR-121 [33] gives

$$\varphi = 0.34\%$$

The flexure steel should be placed normal to the yield lines where possible. The steel layout is shown in Drawing No. 936963 of Appendix E. A check shows that the endwall is satisfactory in bond and pure shear and that no diagonal tension steel is required.

Appendix C

DESIGN OF ENTRANCEWAY

RADIATION DESIGN

Combined Initial Gamma and Neutron Radiation

The shelter entranceway must be designed to reduce the combined dose of initial gamma and neutron radiation to a safe level. At the range for which 100 psi may be expected, the initial radiation level will probably make the entranceway design conservative for residual radiation. Consequently, the entranceway is first designed to reduce initial radiation to a safe level, and then checked to see if residual radiation will present a problem.

A 370-kiloton fission bomb will produce maximum initial gamma radiation at the range at which 100-psi overpressure will occur.[20] The expected maximum initial gamma radiation level at this range is 1×10^5 roentgens; a typical neutron intensity is 1.85×10^5 roentgens.[20] A conservative method for design is to add these two radiation intensities together and consider the sum as initial gamma radiation. Thus the equivalent intensity of initial radiation would be 2.85×10^5 roentgens.

The maximum allowable initial radiation level at the door leading into the shelter proper is 10 roentgens. Thus the attenuation required through the entranceway is

$$A_{req} = \frac{10}{2.85 \times 10^5} = 3.51 \times 10^{-5}$$

The span of the entrance tube is 6-1/2 feet; the rise is 8 feet. For this design the tube is idealized as being rectangular instead of elliptical.

Attenuation of radiation through the entranceway, Drawing No. 936961 of Appendix E, is provided by the two legs and by the blast door with its supporting blast wall. It is assumed that the combination of blast door and blast wall would produce an effect equivalent to a solid wall of steel 1-1/2 inches thick.

For a distributed radiation source the attenuation for the first (exterior) leg is[43]

$$A_1 = \frac{r^2}{2L_1^2}$$

where L is the length of the first leg, and r is the effective radius of the opening and is equal to $\sqrt{A/\pi}$.

With L_1 taken as 24 feet and the area of the opening 52 square feet, the attenuation for the first leg is found to be 1.44×10^{-2}.

One and one-half inches of steel provides shielding equivalent, by ratios of density, to 7.1 inches of 103-pcf soil. The attenuation for the door and blast wall is 2.5×10^{-1}.[21]

Finally, it is necessary to determine the attenuation through the second (inner) leg. The attenuation through this leg is given by[43]

$$A_2 = (1 + T)\, G \left(\frac{w_2}{L_2}\right)^2$$

where T = transmission factor

G = factor which accounts for scattering from prime areas

L_2 = length of second leg

w_2 = width of tube

This formula assumes a square duct, so in this instance w_2 will be taken as 7 feet. With $L_1/w_1 = 3$, $L_2/w_2 = 3$ (taking L_2 as 14 feet), a steel duct and 6-mev gamma we have $G = 0.00960$.[43] Then, neglecting T, we find that A_2 is 2.40×10^{-3}.

The intensity of initial radiation at the blast door would be $2 \times 1.44 \times 10^{-2} \times 2.85 \times 10^5 = 8.2 \times 10^3$ roentgens. The factor of 2 is necessary because both outside legs contribute radiation. The intensity at the shelter would be

$8.2 \times 10^3 \times 2.5 \times 10^{-1} \times 2.40 \times 10^{-3} = 4.9$ roentgens. This level of radiation at the shelter indicates an attenuation for the entire entranceway of 1.72×10^{-5}. This is greater than the required attenuation, so the design is adequate.

Residual Radiation

Considerable variation in the amount of fallout at a range corresponding to 100-psi overpressure may be expected. A maximum fallout rate 1 hour after the explosion is 10,000 roentgens per hour, which indicates an accumulated dose of about 86,500 roentgens for a person in the open for a period of two weeks.

Since the energy level of residual radiation is less than that of initial radiation, it is more easily absorbed by the surrounding media. Therefore the attenuation of residual radiation through the designed entranceway would be greater than that for initial radiation. Hence, since the dose due to residual radiation is less than that for initial radiation, the entranceway is adequate for attenuation of residual radiation.

STRUCTURAL DESIGN

A built-up elliptical underpass section was selected for the entranceway tubing. This tube has a height of 94-1/2 inches and a width of 70-7/16 inches.

The top of the inner tube is 8 feet below ground surface. This amount of cover, assuming 103-pcf soil, would produce a dead load of 820 pounds per square foot. Air-blast-induced ground pressure will be reduced from 100 to 70 psi,[44] which is equivalent to 10,000 pounds per square foot. Using a safety factor of 2, this dynamic load is increased to 20,000 pounds per square foot. Thus an equivalent load of 21,000 pounds per square foot is imposed on the elliptical section.

Using the long dimension of the elliptical tube as the diameter and an allowable stress of 45,000 psi, the hoop stress is found to be 6,860 pounds per inch of structure. The required area is then found to be 0.153 square inch per inch of structure. Therefore, eight-gage plate, with a thickness of 0.164 inch, is adequate. Tests have shown that multiplate conduit of the approximate dimensions of the inner tube will sustain the imposed design loads.[45]

The tubes for the first leg will be subjected to air-induced ground shock from the exterior and to overpressure from the shock wave on the interior. Thus an equalization of pressure will occur, and a minimum-gage tube (No. 12) is selected.

Appendix D

FOUNDATION DESIGN AND PUNCHING ANALYSES

INTRODUCTION

The purpose of this appendix is to develop the relations needed for analysis of an arch foundation and for estimating the amount which a shelter will punch into the soil when it is subjected to a blast load. Both of these analyses are dependent upon the determination of an effective modulus of elasticity of the soil and the derivation of a coefficient, called the coefficient of subgrade reaction, which accounts for the rectangularity of the footing. Since the foundation and punching analyses are dependent upon the effective modulus of elasticity and the coefficient of subgrade reaction, they will be developed first. The approach will be to (1) derive an expression for the effective elastic modulus; (2) derive appropriate equations for the coefficient of subgrade reaction; (3) set down relations for deflections, moments, and stresses which permit determination of the footing reinforcement; and (4) consider the need for diagonal tension steel in the arch, transverse ties between the footings, and torsional reinforcement. This is followed by a dynamic punching analysis based upon a single-degree-of-freedom analog.

The relations derived are for a shelter buried in sand and do not hold for environments consisting of cohesive soils. Further, the calculations are based upon certain fundamental soil parameters which are not well defined. It is considered, however, that the methods presented will provide a conservative design if reasonable values of the soil constants are used.

STATIC DESIGN RELATIONS

Procedure

The relation for the static deflection of a rectangular foundation hinges upon a knowledge of the foundation modulus. It will be shown that the foundation modulus, which is the ratio of the total load on a footing, P, to the total settlement of the footing, δ_o, may be expressed as

$$k_z = \frac{P}{\delta_o} = E'b^2C_{k_z} \qquad \text{(D1)}$$

where E' = increase in modulus of elasticity of the soil per unit depth

b = width of footing

C_{k_z} = coefficient of subgrade reaction

Thus, the foundation modulus depends upon evaluations of E' and C_{k_z}. Accordingly, means of evaluating E' are explained and an equation for C_{k_z} is derived. Then relations for the deflection of the arch are set down together with corresponding equations for longitudinal moments and stresses. This is followed by incidental expressions for the area of reinforcement to resist diagonal tension. Finally, the requirements for transverse ties and torsional steel are considered.

Determination of Foundation Modulus

Evaluation of E' in the expression for the foundation modulus, Equation D1, is accomplished with the aid of plate bearing tests by (1) determining k_z from the load-settlement curve of the plate, (2) deriving an appropriate relation for C_{k_z} for the plate, and (3) substituting k_z, C_{k_z}, and the plate dimensions in Equation D1. Equation D1 can then be solved for E'. Assuming that the value of E' is constant for a given soil, k_z can be found for a particular foundation upon deriving the corresponding value of C_{k_z}.

The coefficient of subgrade reaction, C_{k_z}, is a constant which accounts for the shape of the footing and the variation of the modulus of elasticity with depth. Derivation of a relation for C_{k_z} is contingent upon obtaining an expression for this variation.

An expression for the change of the modulus of elasticity with depth is obtained by assuming that the modulus of elasticity of the soil is a linear function of depth. This assumption is valid for loose sands and for dense sands with large lateral confining pressure and is approximately correct at relatively shallow depths for intermediate confinement. Employing the stated assumption and replacing the unit pressure under the footing, q, by an equivalent depth of soil, h, as shown in Figure D1, the effective modulus of elasticity becomes

$$E = E'\left(\frac{q}{\rho} + z\right) = E'(h + z) \qquad \text{(D2)}$$

where ρ is the density of the soil and z is any arbitrary depth within the soil mass below the footing (Figure D1). It has been found from experience that reasonable values of E are obtained if the value of q in Equation D2 is taken as the value of the unit load at the upper limit of the elastic region of behavior in the load-settlement curve of the plate bearing test.

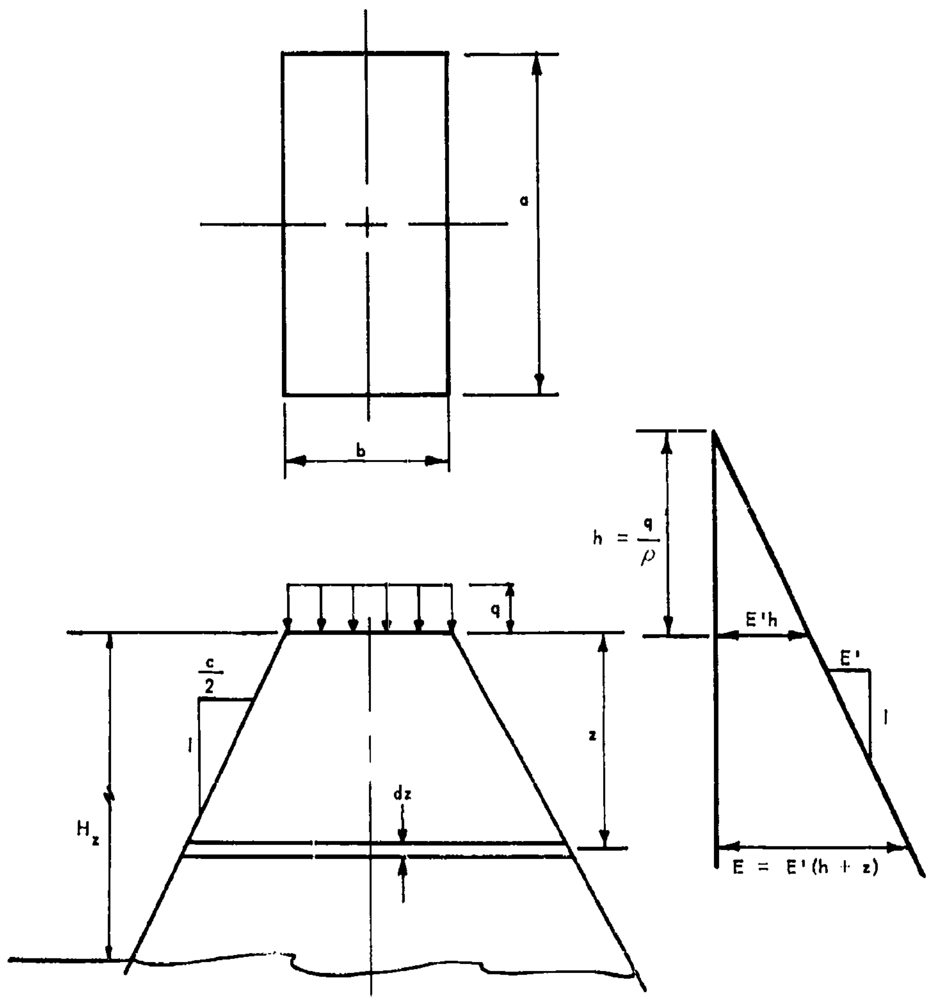


Figure D1. Increase in elastic modulus due to uniform surface load.

Equation D2 is used in the development of Equation D1 as follows: The settlement beneath the contact face of the footing, Drawing No. 936963, is equal to the compression of the truncated pyramid which extends a depth, H_z, to bedrock.[46] This is the sum of the compressive strains of all the successive horizontal layers dz of the pyramid. The total pressure on the differential strip is

$$P = AE\epsilon = (a + cz)(b + cz) E \frac{d\delta}{dz} \tag{D3}$$

Where $d\delta$ is the unit deformation in the elemental length dz. Substituting Equation D2 into Equation D3 and solving for the unit deformation,

$$d\delta = \frac{P}{E'(a + cz)(b + cz)(h + z)} dz \tag{D4}$$

The total deformation of the contact surface of the footing is

$$\delta_o = \int_o^{H_z} d\delta = \frac{P}{E'b^2} \int_o^{cH_z/b} \frac{dz'}{(r + z')(1 + z')(s + z')} \tag{D5}$$

where $z' = cz/b$

$r = a/b;\ a \geq b$

$s = ch/b$

If the integral is defined as $1/C_{k_z}$, Equation D5 may be written as

$$\frac{P}{\delta_o} = E'b^2 C_{k_z} = k_z$$

which is the form of Equation D1. On integration, the coefficient of subgrade reaction, C_{k_z}, becomes

$$C_{k_z} = \frac{1}{(r-1)(r-s)(s-1)}\left[(s-1)\ln\left(r+\frac{cH_z}{b}\right) + (r-s)\ln\left(1+\frac{cH_z}{b}\right) - (r-1)\ln\left(s+\frac{cH_z}{b}\right) - (s-1)\ln r + (r-1)\ln s\right] \qquad \text{(D6)}$$

A study of Equation D6 has shown that H_z may be set equal to infinity without gross loss of accuracy provided that a rigid interface does not lie within 10 feet beneath the bottom face of the footings. If H_z is set equal to infinity in the upper limit of integration (Equation D5), and c is taken as unity (in accordance with the conditions of the Boussinesq Equation), C_{k_z} may be expressed as

$$C_{k_z} = \frac{r-s}{\dfrac{\ln s}{s-1} - \dfrac{\ln r}{r-1}} \qquad r \neq s \neq 1$$

$$C_{k_z} = \frac{s-1}{1-\dfrac{\ln s}{s-1}} \qquad r = 1,\ s \neq 1 \qquad \text{(D7)}$$

Equation D7 with Equation D1 permits determination of the foundation modulus for any specific site condition or for any specific footing dimension.

Determination of Deflections, Moments, and Stresses

The arch is treated as a beam on an elastic foundation (Figure D2) to permit evaluation of the diagonal tension stresses which have been found from tests to extend into the arch approximately one and one-half radii from the endwalls.[4] Such treatment also enables calculation of the deflection of the mid-span of the arch with respect to the end foundations, and permits determinations of the longitudinal moments, flexural stresses, and shear stresses.

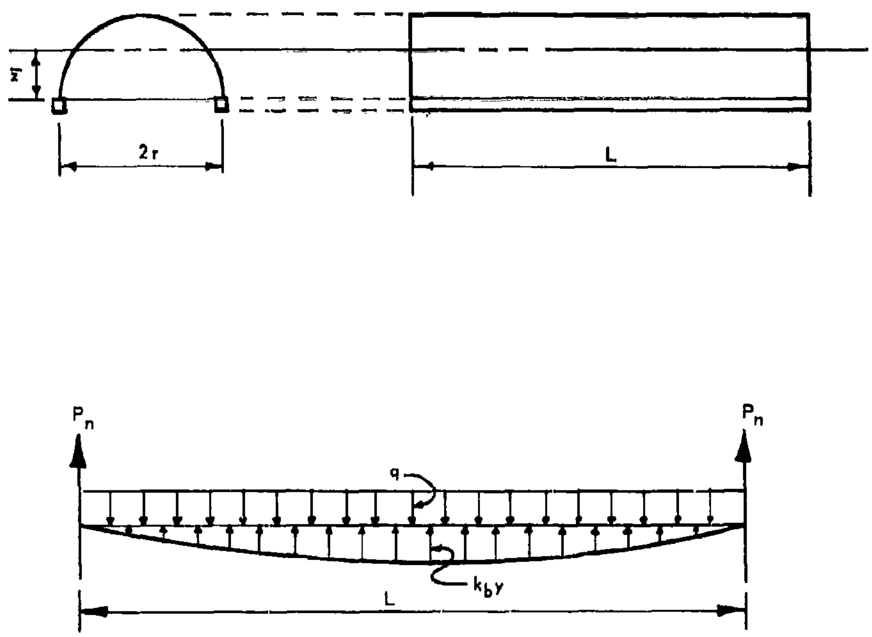


Figure D2. Arch as a beam on an elastic foundation.

The assumed loading on the arch is shown in Figure D3. This loading, as stated previously, has been found to be approximately equal to the load at the time of maximum response of an actual structure. From the assumed loading the vertical arch reaction may be derived and applied (as a uniformly distributed load) to the arch as a beam on an elastic foundation. The vertical reaction for one-half the arch is found to be

$$V_a = \frac{7}{8} p_o r \qquad \text{(D8)}$$

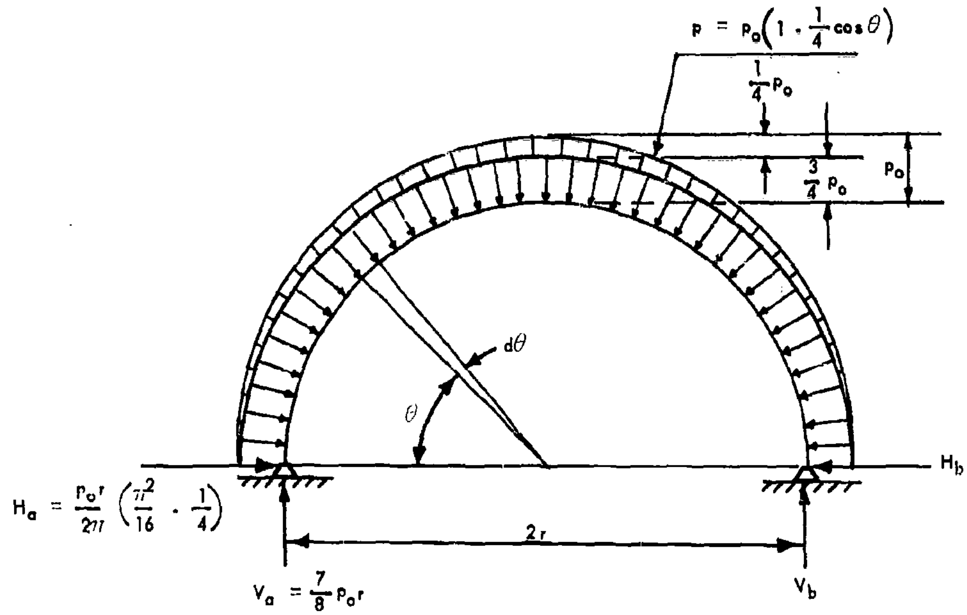


Figure D3. Arch loading diagram.

Also, there is assumed to be a uniformly distributed uplift pressure acting on the base of the footing from the free field overpressure equal to

$$u = p_o b \tag{D9}$$

The net uniformly distributed load on the beam per unit length is, therefore,

$$q_b = 2p_o\left(\frac{7}{8}r - b\right) \tag{D10}$$

The uplift acting on the endwall footing may be considered to result in an upward concentrated load on each end of the "beam" equal to

$$P_n = 2p_o br \tag{D11}$$

With the given loading, the deflection and the bending moment at mid-span of the "beam" are[47]

$$y_c = \frac{q_b}{k_b} - \frac{\frac{4P_n\lambda}{k_b}\cosh\frac{\lambda L}{2}\cos\frac{\lambda L}{2}}{\sinh\lambda L + \sin\lambda L} \quad \text{(D12)}$$

and

$$M_c = \frac{2P_n}{\lambda}\,\frac{\sinh\frac{\lambda L}{2}\sin\frac{\lambda L}{2}}{\sinh\lambda L + \sin\lambda L} \quad \text{(D13)}$$

where q = unit load, lb/in.

k_b = foundation modulus, lb/in.^{2}

$\lambda = [k_b/(4E_cI_n)]^{1/4}$

E_cI_n = beam stiffness factor

L = beam length

$k_b = 2k_z/L$, where k_z is obtained from Equation D1

The properties of the "beam" required for evaluation of the deflection, moment, and stresses are found with the aid of Figure D4 as follows:[48]

$$\bar{z} = \frac{tr^2\sin\theta + \left(r\cos\theta - \frac{H}{2}\right)Hb}{rt\theta + Hb} \quad \text{(D14)}$$

$$I_{s_o} = 2tr^3\left(\frac{\theta}{2} + \frac{1}{4}\sin 2\theta\right) \quad \text{(D15)}$$

$$I_{B_o} = \frac{2bH^3}{12} + 2bH\left(r\cos\theta - \frac{H}{2}\right)^2 \quad \text{(D16)}$$

$$I_o = I_{s_o} + I_{B_o} \tag{D17}$$

and I_n about the neutral axis, calculated by the Parallel Axis Theorem, is

$$I_n = I_o - 2(rt\theta + Hb)\bar{z}^2 \tag{D18}$$

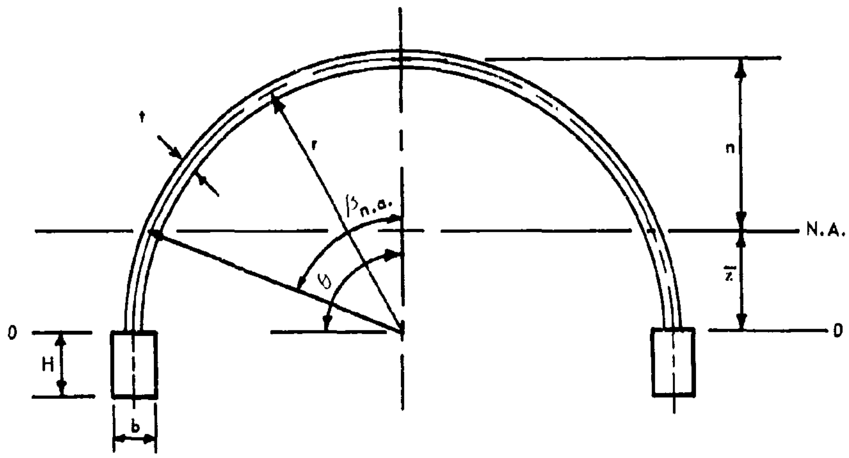


Figure D4. Sketch for determining beam properties.

The maximum longitudinal compressive force N_x at the extreme fibre of the "beam" is

$$N_x = -\frac{M_c n t}{I_n} \tag{D19}$$

where $n = r - \bar{z}$

The maximum compressive stress f_c due to N_x is

$$f_c = \frac{N_x}{t} \tag{D20}$$

The maximum tensile stress, f_t, within the footing is

$$f_t = \frac{M_c (\bar{z} + H)}{I_n} \qquad \text{(D21)}$$

Reinforcing steel to resist longitudinal compressive stresses within the "beam" should be provided in accordance with the calculated stresses and should be sized in accordance with allowable static design stresses. The minimal reinforcement provided, however, should not be less than that required for temperature and shrinkage stresses.

Reinforcing steel within the footings to resist f_t (Equation D21) should be provided in accordance with the design stresses in Table II.

If the concrete carries no shear the unit shear stress to be carried by the reinforcement is[49]

$$v_{max} = \frac{V}{2tr} \frac{\beta_{n.a.}}{\sin \beta_{n.a.}} \qquad \text{(D22)}$$

where $\beta_{n.a.}$ is the angle which gives the location of the neutral axis of the arch cross section. V equals the maximum shear at the inner faces of the endwalls and is given approximately by the expression

$$V = 2 p_o r (b - D_{ew}) \qquad \text{(D23)}$$

For a given spacing, s, the area of diagonal tension steel required is

$$A_v = \frac{stv_{max}}{f_y} \qquad \text{(D24)}$$

All that remains of the static analysis is to consider the requirements for transverse ties and torsional steel.

Requirement for Transverse Ties

Lateral displacement of the footings must be restricted. The reason is that bending stresses within an arch having free-sliding abutments may be 200 percent greater than those for a hinged arch. To assure against these bending stresses it is necessary to investigate the requirement for transverse ties (ties between the footings).

The primary forces acting on the footings are shown in Figure D5, in which

p_p = passive lateral pressure

p_a = lateral hydrostatic pressure

S_f = frictional forces

V_a = vertical component of arch reaction

N = resultant of soil pressure forces

F = D'Alembert's inertia force

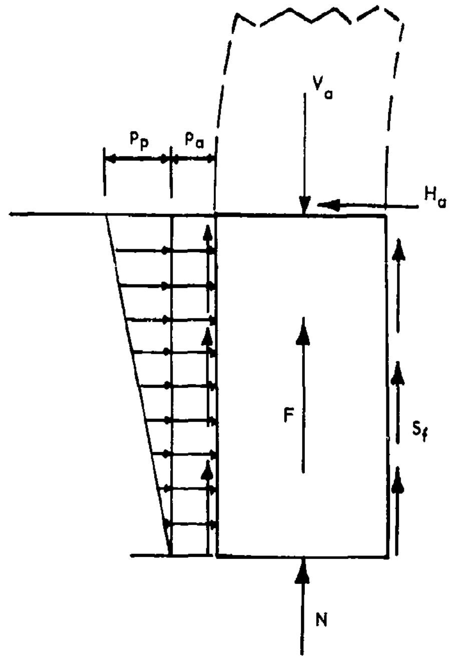


Figure D5. Foundation forces.

It is readily shown for the arch of Appendix A that if the depth of footing is 3 feet or greater, the summation of lateral forces, p_a, is slightly greater than the horizontal reaction, H_a, and lateral ties are not required.

Footing Torsion

Rotation of the footing has produced failure of the foundation in test structures.[6] Since considerable restraint is developed in monolithic construction at the junction between the longitudinal and transverse footings and the endwalls, torsional stresses induced by restraint deserve careful consideration.

Determination of the actual torsional stresses induced in the footings is difficult because the loading and restraint in the foundation are complex. The maximum probable torsional stress in the foundation can be found, however, by assuming that the torque at the junctions between the footings and endwalls is a function of the rotation occurring at the springing:[50]

$$T_e = \theta_e b_e^3 t_e C^{-1} \qquad \text{(D25)}$$

where $\theta_e = 0.00473 (P_o r^3/EI)$ = rotation at the support of the hinged-end arch (Figures D3 and D5)

b_e = width of rectangular section, inches

t_e = depth of rectangular section, inches

C = detrusion coefficient obtained from test data

The actual footing has an isosceles trapezoidal section, and therefore it is replaced in the computations by an equivalent rectangular section of width b_e.[51]

For economy, the term T_e must be equal to the torque at cracking defined by the relation

$$T_e = \alpha b_e^2 t_e v_{max} + \frac{\lambda}{2} b' t' \frac{A_v}{s} f_v \qquad \text{(D26)}$$

in which, numerical values of the elastic constants, α and λ, are obtained from Table DI. The remaining terms in Equation D26 are defined in the List of Symbols. By equating Equations D25 and D26, the ratio of the area of stirrups to the spacing of stirrups, A_v/s, may be obtained. Such computations demonstrate that a hinged joint should be provided in the top of the footing. With a hinged joint, the resistance of the footing is compared with the torque developed by the external force couple, the Σp_a and H_a (Figure D5), to determine if the footing has adequate torsional strength.

Table DI. Numerical Values of the Principle Elastic Torsion Parameters

$\frac{t_e}{b_e}$ or $\frac{t'}{b'}$	1.0	1.5	2.0	2.5	3.0
α	0.208	0.231	0.246	0.258	0.267
λ	1.669	1.599	1.614	1.654	1.689

DYNAMIC ANALYSIS

Procedure

The rigid-body motion of a buried arch can be predicted from the response of a mechanical analog. The analog utilized is a single-degree-of-freedom system for which equations and response charts are readily available in the literature.[52] In this section, the response equation is given and the load-time and load-deflection relations are presented.

Equation of Motion

The differential equation of motion for the single-degree-of-freedom is

$$m \frac{d^2y}{dt^2} + c \frac{dy}{dt} + K_z y = f(t) \qquad \text{(D27)}$$

where m = mass of system

c = damping coefficient

K_z = spring constant

$f(t)$ = load function

The mass, m, in the analogous system is taken as the mass of the arch, the endwalls, and the earth directly over the shelter. The damping is assumed to be zero.

The solution to Equation D27 depends upon the resistance, $K_z y$, and the forcing function, $f(t)$. For a buried arch subjected to blast loading, the idealized resistance functions of Figure D6 may be used.

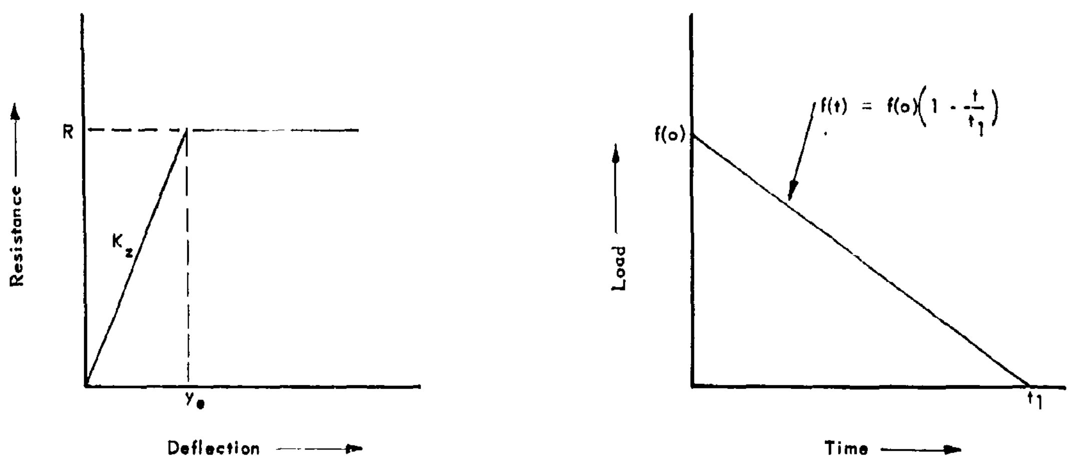


Figure D6. Idealized resistance and load functions.

The peak ordinate of the bilinear resistance function is the ultimate resistance of the footings to punching shear. Considering only the longitudinal footings (for reasons which will become evident),

$$R = 2blq_d$$

where b = width of footing

l = length of shelter

q_d = ultimate punching shear strength of the soil

The slope of the resistance curve is $K_z = 2k_z$, where k_z is determined from Equation D1. The factor of 2 accounts for the two longitudinal footings; in using Equation D1, k_z must be evaluated for the dimensions of a single footing. It is acceptable to use only the resistance of the longitudinal footings in the analogous system provided that the load function is taken as twice the thrust in the arch at the footings.

The load function, $f(t)$, taken as twice the thrust in the arch, is based on an assumed radial loading of peak intensity equal to p_o distributed uniformly about the periphery of the arch. Considering that there are two longitudinal footings, the peak value of the load to be used in the analogous system is

$$f(o) = 2p_o rl$$

The effective duration of the load may be obtained from the semi-empirical relation[53]

$$t_1 = 0.40\left(\frac{100}{p_o}\right)^{0.6} W^{1/3}$$

where t_1 = effective duration of the blast, sec

p_o = peak overpressure on the surface, psi

W = yield of the weapon, megatons

With the preceding parameters, the load and resistance functions are completely defined.

Equation D27, with the load and resistance functions of Figure D6, has been solved and the results are readily available in the form of response charts. It is a simple matter to determine the peak deflection of the mass in the analogous system from these charts. The deflection of the analogous system will be the same as the deflection of the arch foundation.

EXAMPLE COMPUTATIONS

Use of the relations presented in this appendix are best illustrated by the computations of Table DII. The problem is to design the footings for the 20-foot by 48-foot shelter shown in the design drawings of Appendix E. The footing size is determined by the allowable deflection and not by bearing capacity. The shelter is designed to withstand 100 psi from a megaton explosion, i.e., a long-duration blast. The earth foundation and backfill material are sand possessing the following properties:

$$\text{density} = 120 \text{ lb/ft}^3$$

$$\text{foundation modulus} = 258.3 \text{ lb/in.}^3 \text{ (plate)}$$

The foundation modulus was obtained from 12-inch by 12-inch plate-bearing test data. Other soil characteristics may require design modifications.

It is required to determine: (1) the foundation modulus for the shelter footings; (2) the deflection of the center of the shelter with respect to the endwalls; (3) the flexural and shear stresses and the corresponding flexural and shear steel; (4) the amount of torsional steel required; and (5) the amount that the arch punches into the soil.

The necessary design sketch is given in Figure D7; the calculations proceed as in Table DII.

Design Data

$r = 10'$

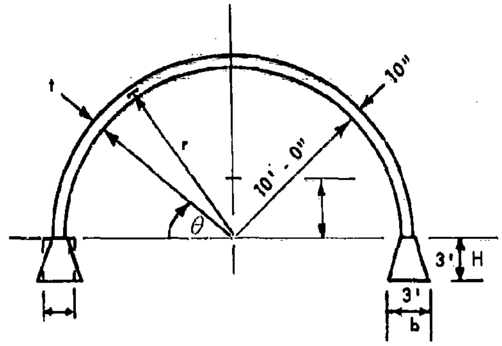
$t = 10''$

$H = 3.0'$

$b = 3.0'$

$f'_c = 4{,}000 \text{ lb/in.}^2$

$E_c = 1{,}800{,}000 + 460 f'_c$



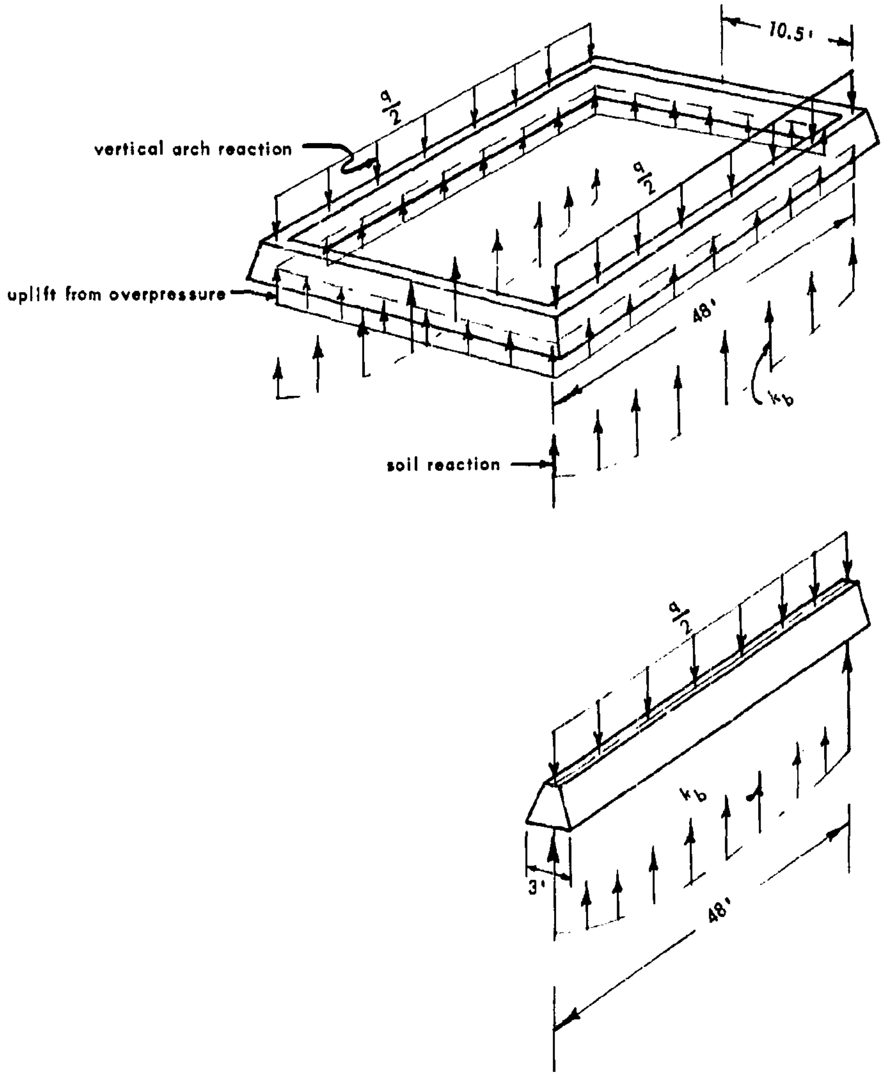


Figure D7. Sketch of loading on foundation.

Table DII. Example Computations

Computations	Results	Comments
For the foundation modulus:		
Density of soil = 120 lb/ft^3		Determined from Laboratory test.
Foundation modulus, k_o = 258.3 $lb/in.^3$		Determined from plate-bearing test data.
1. Determine E' from bearing tests on a 12-in. x 12-in. plate and Equation 1, as follows:		$E' = k_p/b^2 C_{k_z}$
$k_p = A_p k_o = 144\ in.^2 \times 258.3\ lb/in.^3$	37,200 lb/in.	
From Equation D7		
$C_{k_z} = \dfrac{s - 1}{1 - \dfrac{\ln s}{s - 1}} = \dfrac{15.60 - 1}{1 - \dfrac{\ln 15.6}{15.6 - 1}}$	18.0	C_{k_z} for 12-in.-sq plate and soil media of infinite depth.
where $s = \dfrac{ch}{b} = \dfrac{1 \times 15.6}{1}$	15.6	
$c = 1$		
$h = \dfrac{q}{\rho} = \dfrac{13\ lb/in.^2 \times 144\ in.^2/ft^2}{120\ lb/ft^3}$	15.6 ft	q taken at limit of elastic range. The shear value of the soil at the limit of the elastic range, determined from the 12-in.-sq plate-bearing test, is 13 $lb/in.^2$

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
therefore, from Equation D1, $E' = \frac{k_p}{b^2 C_{k_z}} = \frac{37,200}{12^2 \times 18.0}$	14.4 lb/in.^{3}	
2. Assume a footing width of 3 ft and find the ultimate shear strength, q_d, for the soil under the concrete footings.[51]		
With $\phi = 35^o$, and $\rho = 120$ lb/ft^3, it is found from Ref. 6 that $q = \rho(D_f N_q + 0.5 b N_\gamma)$		Surcharge $\pm$12-ft depth of soil, or 10-psi pressure.
which, for the prevailing surcharge, gives $q = 43 + 1.6b$ $= 43 + 1.6 \times 3$	47.8 kips/ft^2	
3. Find equivalent surcharge, h, due to q: $h = \frac{q}{\rho} = \frac{47,800 \text{ lb/ft}^2}{120 \text{ lb/ft}^3}$	398.0 ft	Equivalent height of soil

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
4. Compute C_{k_z} and k_z for footing from Equations D7 and D1:		
$C_{k_z} = \dfrac{16 - 132.7}{\dfrac{\ln 132.7}{132.7 - 1} - \dfrac{\ln 16}{16 - 1}}$	792	
where $r = \dfrac{a}{b} = \dfrac{48}{3}$	16.0	
$s = \dfrac{ch}{b} = \dfrac{398}{3}$	132.7	
then $k_z = E'b^2 C_{k_z}$		Equation D1.
$k_z = 14.5 \times 36^2 \times 792$	14.88×10^6 lb/in.	
and $K = \dfrac{k_z}{A}$		
$K = 14.883 \dfrac{10^6}{3 \times 48 \times 144}$	718 lb/in.^3	Area of footing = $3 \times 48 \times 144\ \text{in.}^2$

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
<u>For the deflection at the center of shelter foundation</u>		
1. Find the properties of the "beam," as follows:		
$\bar{z} = \dfrac{0.833 \times 10.5^2 + \left(0 - \frac{3}{2}\right) 3 \times 3}{10.5 \times 0.833 \frac{\pi}{2} + 3 \times 3}$	3.40 ft	Equation D14. Area of an equivalent rectangular footing having a width of 1.91 ft and depth of 3.0 ft could have been used.
$I_{s_o} = 2 \times 0.833 \times 10.5^3 \times \frac{\pi}{4}$	1,515 ft^4	Equation D15. "I" of arch about its base.
$I_{B_o} = 2\left[3 \times \frac{3^3}{12} + 3 \times 3\left(0 - \frac{3}{2}\right)^2\right]$	54.0 ft	"I" of footings about the top of the footings.
$I_o = 1{,}515 + 54.00$	1,569 ft^4	"I" of X-Sect.
$I_n = 1{,}569 - 2\left(10.5 \times 0.833 \times \frac{\pi}{2} + 3 \times 3\right) 3.45^2$	1,027 ft^4	Equation D18. "I" at N.A. of X-Sect.

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
2. Find the characteristic length, λ, of the beam.[47] Refer to computations for Equation D1 in which K was found to be equal to 718 $lb/in.^3$ "$E_c I_n$" is based on the X-Sect. of the arch which includes the footings. The foundation modulus, K, is modified, therefore, to include the effect of two footings:		
$k_b = 2bK = 2 \times 36 \times 718$	51,700 $lb/in.^2$	
Then $\lambda = \left(\frac{k_b}{4E_c I_n}\right)^{1/4}$		
$\lambda = \left(\frac{51.700 \times 10^3}{4 \times 3.64 \times 10^6 \times 1,027}\right)^{1/4}$	5.08×10^{-3} $in.^{-1}$	
in which		
$E_c = 1,800,000 + 460 f'_c$	3.64×10^6 $lb/in.^2$	$f'_c = 4,000$ psi

Table D11. Example Computations (Cont'd)

Computations	Results	Comments
3. Find the loading on the beam:		See Figure D10.
$q_b = 2 \times 100\left(\frac{7}{8} \times 10.5 - 3\right) \frac{12}{1,000}$	14.86 kips/in.	Equation D10.
$P_n = 2 \times 100 \times 36 \times 10.5 \times 12$	907.5 kips	Equation D11.
Therefore,		
$y_c = \frac{14.86}{51.7}\left(\frac{4 \times 907.5 \times 5.08 \times 10^{-3}}{51.7}\right) \left(\frac{2.29 \times 0.1006}{9.34 + 0.210}\right)$	0.278 in.	Equation D12.
The foundation pressure, p_f, beneath the footing is found as follows:		
$p_f = Ky_c = 718 \times 0.278$	200 lb/in.^{2}	
The ultimate shear value, q, was computed previously as 47,800 lb/ft^2, which is equal to 322 lb/in.^{2} The ratio of q to p_f is		Equation D1 computations.
$\frac{q}{p_f} = \frac{322}{200}$	1.61	Conservative design.

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
For the flexural and shear stresses and corresponding steel:		
1. Find the bending moment at the center of the beam:		
$M_c = \frac{2 \times 907.5}{5.08 \times 10^{-3}} \left(\frac{2.06 \times 0.995}{9.34 + 0.210}\right)$	76,700 in.-kips	Equation D13.
2. The maximum tensile stress, f_t, is		
$f_t = \frac{7.67 \times 10^4 \times 6.4 \times 12}{1,027 \times 12^4}$	280 lb/in.^{2}	Equation D21. Stress at the bottom face of the footing.
and the stress at the top of the footing is		
$f_t = 280 \frac{41.4}{77.4}$	150 lb/in.^{2}	See sketch below.

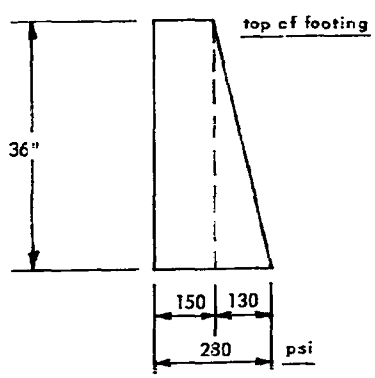


Table DII. Example Computations (Cont'd)

Computations	Results	Comments
To find the area of steel required to resist the tensile stresses, the footing is divided as shown in the sketch below, and the reinforcing steel in each division is determined as follows:		The tensile force in the division is equal to the product of the average stress and the area.

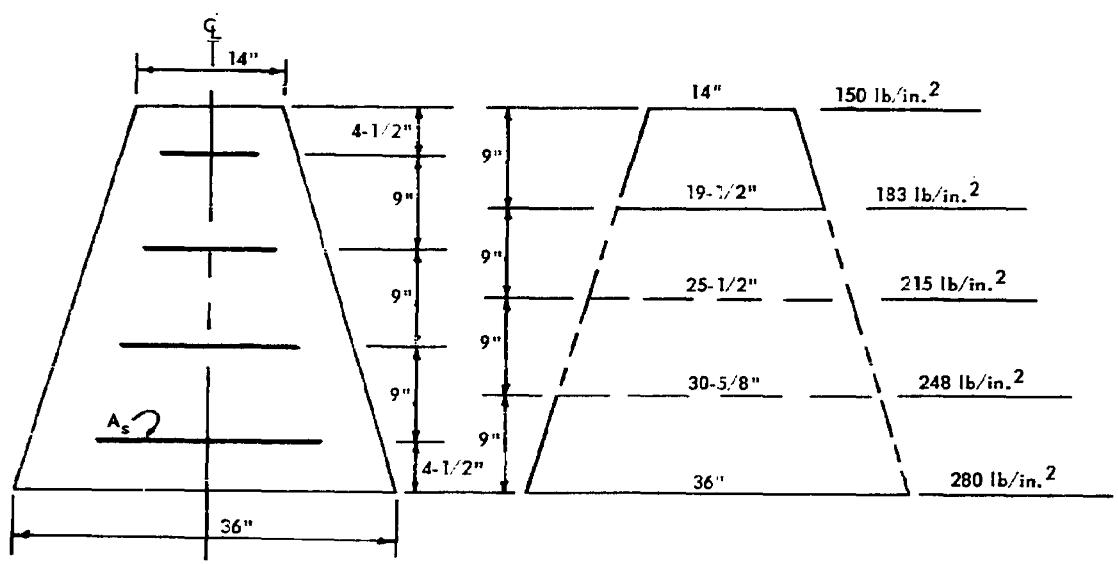


Table DII. Example Computations (Cont'd)

Computations	Results	Comments
The tensile force in the top 9-in. depth of footing is		
$F_{t_t} = \frac{150 + 183}{2} [4.5 (14 + 19.5)]$	25,200 lb	
and that in the bottom 9-in. depth is		
$F_{t_b} = \frac{248 - 280}{2} [4.5 (30.63 + 36)]$	79,200 lb	
Therefore, the required area of steel is		
$A_{s_t} = \frac{25,200}{40,000}$	0.63 in.^{2}	Use two #6 bars.
$A_{s_b} = \frac{79,200}{40,000}$	1.98 in.^{2}	Use five #6 bars. The steel for the remaining divisions is obtained by direct proportion.

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
For the torsional stress and corresponding steel:		
$\theta_e = \dfrac{4.73 \times 10^{-3} \times 100 \times 126^3}{3.03 \times 10^8}$	3.14×10^{-3}	Equation D25. Rotation of springing of the hinged-end arch.
where $I = \dfrac{bt^3}{12} = \dfrac{1 \times 10^3}{12}$	83.4 in.^{4}	
$E_c I = 3.64 \times 10^6 \times 8.34 \times 10$	3.03×10^8	$E_c = 1,800,000 + 460 f'_c$
By assuming a factor of safety of unity, θ_p may be set equal to θ_e. Then, the torque, T_e, may be determined as follows:		
$T_e = b_e^3 t_e \theta_p C^{-1}$	1.38×10^7 in.-lb.	Equation D26.
$= \dfrac{23^3 \times 36 \times 3.14 \times 10^{-3}}{100 \times 10^{-6}}$		
where b_e is the width of an equivalent rectangular beam of width and depth equal to 23 in. $\times$ 36 in. The detrusion coefficient, C, is assumed equal to 100×10^{-6}. But from Equation D26		Equivalent beam having the same depth and cross-sectional area as the original beam. Reference 48.
$T_e = \alpha b_e^2 t_e v_{max} + \dfrac{\lambda}{2} b't' \dfrac{A_v}{s} f_v$		Equation D26.

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
in which α, a constant of the elastic theory of torsion, is obtained from Table DI.[47]		
For the equivalent beam considered, the following numerical values are obtained:		
$\alpha = 0.231$		
$\lambda = 1.599$		
$t_e = 36$		
$t' = 33.5$ (assumed)		Assumed 2-1/2-in. concrete cover over the reinforcing steel.
$b_e = 23$		
$b' = 18$ (assumed)		
and $v_{max} = \frac{4.54 \times 10^5}{2 \times 23 \times 126} \left(\frac{1.237}{0.945}\right)$	103 lb/in.^{2}	Equation D22.
where $V = 4.54 \times 10^5$ lb		Equation D23.
$\beta_{n.a.} = 70^\circ 51' = 1.237$ radians		

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
The tensile stress, f_v, in the diagonal tension reinforcement is		
$f_v = \frac{E_s}{E_c} v_{max} \sqrt{2} = \frac{30 \times 10^6}{3.64 \times 10^6} \times 103 \times 1.414$	1,200 lb/in.^{2}	
and the ratio A_v/s obtained from Equation D26 is		
$1.38 \times 10^7 = 0.231 \times 23^2 \times 36 \times 103 + \frac{1.599}{2} \times 18 \times 33.5 \frac{A_v}{s} \times 1{,}200$		
or $\frac{A_v}{s} = 23.1$		
The computed ratio, A_v/s equal to 23.1, presents an impossible condition to satisfy with conventional stirrups. If v_{max} had been assumed equal to $0.1 f_c'$ and f_v equal to 40,000 psi, A_v/s would be equal to 0.62, which cannot be easily satisfied. Therefore, at the top of the footing, a hinged joint is provided and the torsional resistance of the footing is compared to the applied torque, as follows:		
$H_a = \frac{100 \times 126}{6.28} \left(\frac{9.82}{16} - \frac{1}{4} \right)$	720 lb/in.	See Figure D3.
$\Sigma p_a = 0.2 p_o H = 0.2 \times 100 \times 36$	720 lb/in.	See Figure D5. p_a equal to $0.2 p_o$ has been discussed previously.

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
The torque due to the external couple formed by the Σp_a and H_a is		
$T = \frac{H}{2} \Sigma p_a = \frac{36}{2} \times 36 \times 20$	12,960 in.-lb	
The resistance of the footing to torque is obtained from the first part of Equation D26:		
$T_r = \alpha b^2 t_e v_{max}$		
and for v equal to $0.03 f'_c$		Allowable design stress for $f'_c = 4,000$ psi.
$T_r = 0.231 \times 23^2 \times 36 \times 102$	528,000 in.-lb	
Since T_r is greater than T, the footing design is satisfactory in torsion.		
For the punching displacement of the footing through the soil:		
$m \frac{d^2y}{dt^2} + K_z y = f(t)$		
where m = weight of concrete + weight of backfill + weight of soil beneath footings moving with the structure		For this computation, the total mass of the soil-structure is used.
$K_z = 2k_z$		Equation D13. Structure is considered to move as a rigid mass. Therefore, the total length of the shelter is considered.
f(t) = footing reaction expressed as a function of time		

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
Weight of shelter (Appendix E):		
Area of X-Sect. $= \frac{0.7854}{2}(21.66^2 - 20.00^2) + 2 \times 2 \times 3$		Shell + footings.
$= 32.60 \text{ ft}^2$		
Weight of X-Sect. $= 32.60 \times 1 \times 150 \text{ lb/ft}^3$	4,890 lb/ft	
Total weight of arch $= 4,890\left(\frac{48}{1,000}\right)$	234.7 kips	
Weight of 18-in.-thick endwalls $= 0.7854 \times 10^2 \times 1.5 \times 150$	17.7 kips	Two endwalls.
Weight of backfill $= 48\left(15.83 \times 21 - \frac{0.7854 \times 20.84^2}{2}\right)110$	855.2 kips	Five-ft earth cover over the crown.
Assume a rectangular wedge of earth 3 ft wide × 3 ft deep beneath the footing moves with the structure:		The true apparent mass is unknown. Reference 43.
The weight of the wedge beneath the two footings is		
$18 \times 48 \frac{110}{1,000}$	95.0 kips	
Total weight:	1,202 kips	

Table DII. Example Computations (Cont'd)

Computations	Results	Comments
From Equation D1, $k_z = 14.883 \times 10^6$ lb/in.		Equation D1 computations.
and $K_z = 2k_z = 29.766 \times 10^6$ lb/in.		
Load function, f(t): Assuming a radial loading of intensity p_o, the maximum thrust, T, within the arch is		Two footings require a spring constant equal to $2k_z$.
$T = rp_o = \frac{10.5 \times 12 \times 100 \times 12}{1,000}$		
$= 151.2$ kips/ft		
where $r = 10.5$ ft		
$p_o = 100$ lb/in.^{2}		
The total footing reaction, T_t, is therefore equal to 2T times the length of the footings:		
$T_t = 2 \times 151.2 \times 48$	14,510 kips	

Table DII. Example Computations (Cont'd)

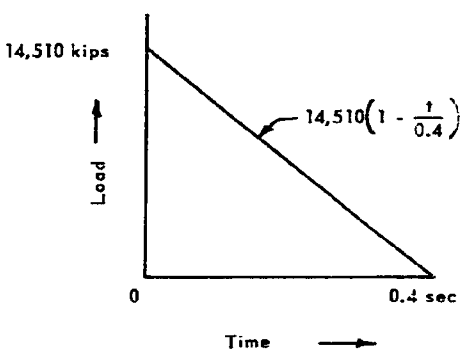
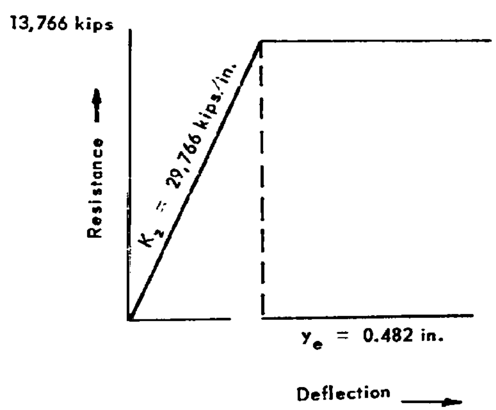
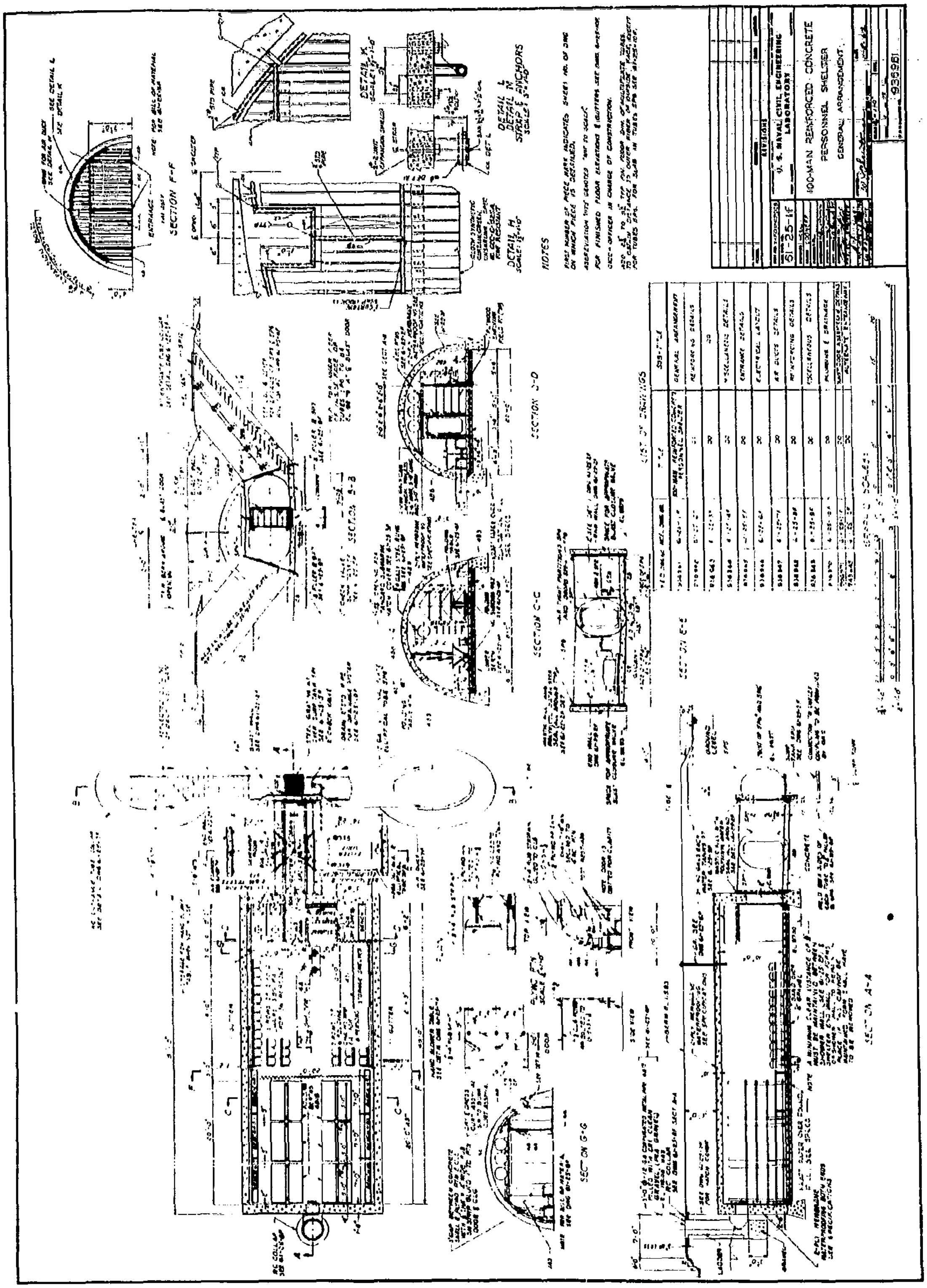
Computations	Results	Comments
Load and resistance functions: The load-time curve and load-deflection curve for the system are given in the sketches below.  Load-time curve (t_1 for 1 MT).  Resistance-deflection curve.		Load-deflection curve: See computation with Equation 13. q_d = 47.8 kips/ft^2. The unit load causing this pressure is $2bq_d$ = 6 x 47.8 = 286.8 kips/ft. The total load on the footings is therefore $2bq_dL$ = 286.8 x 48 = 13,766 kips.

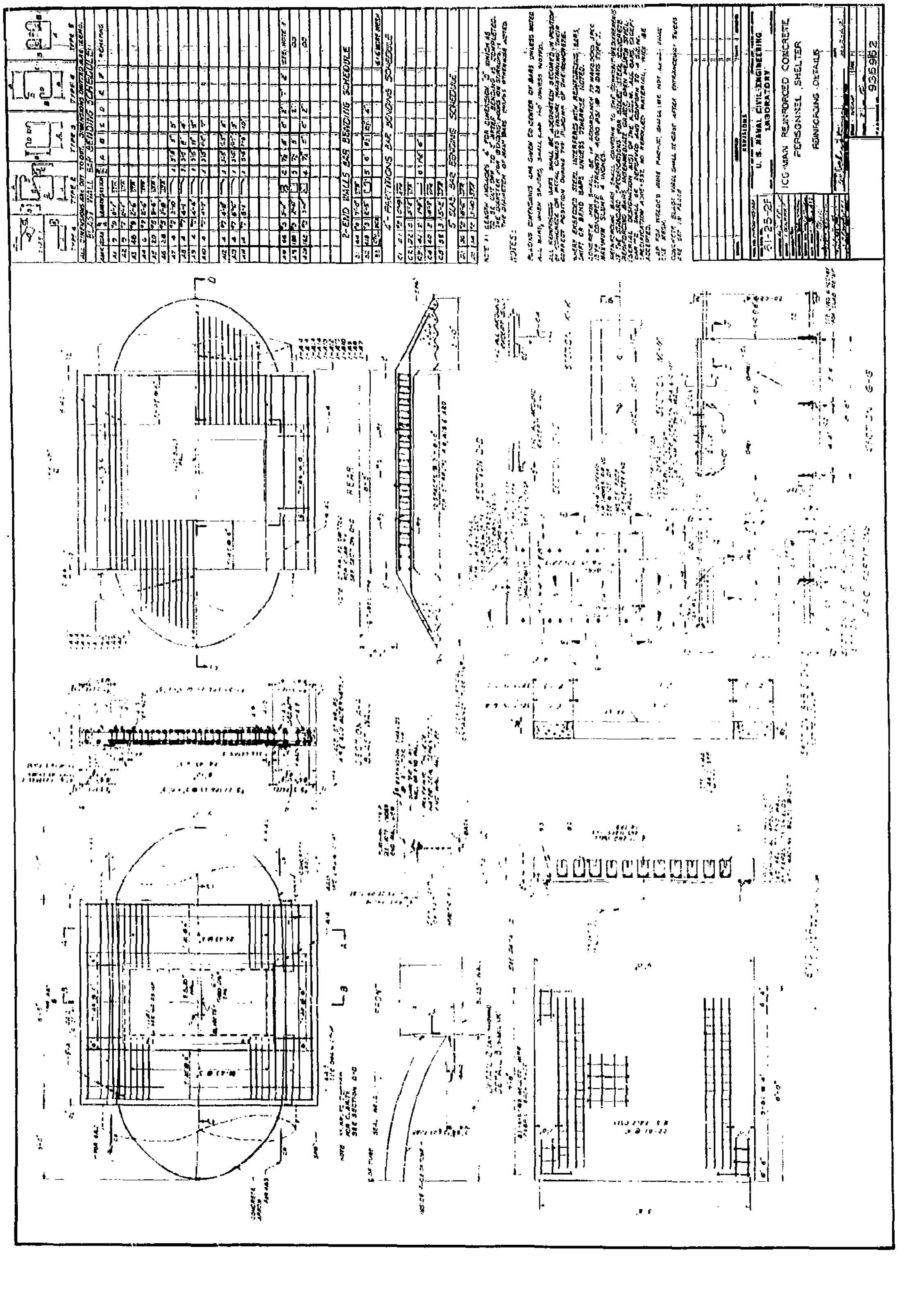
Table DII. Example Computations (Cont'd)

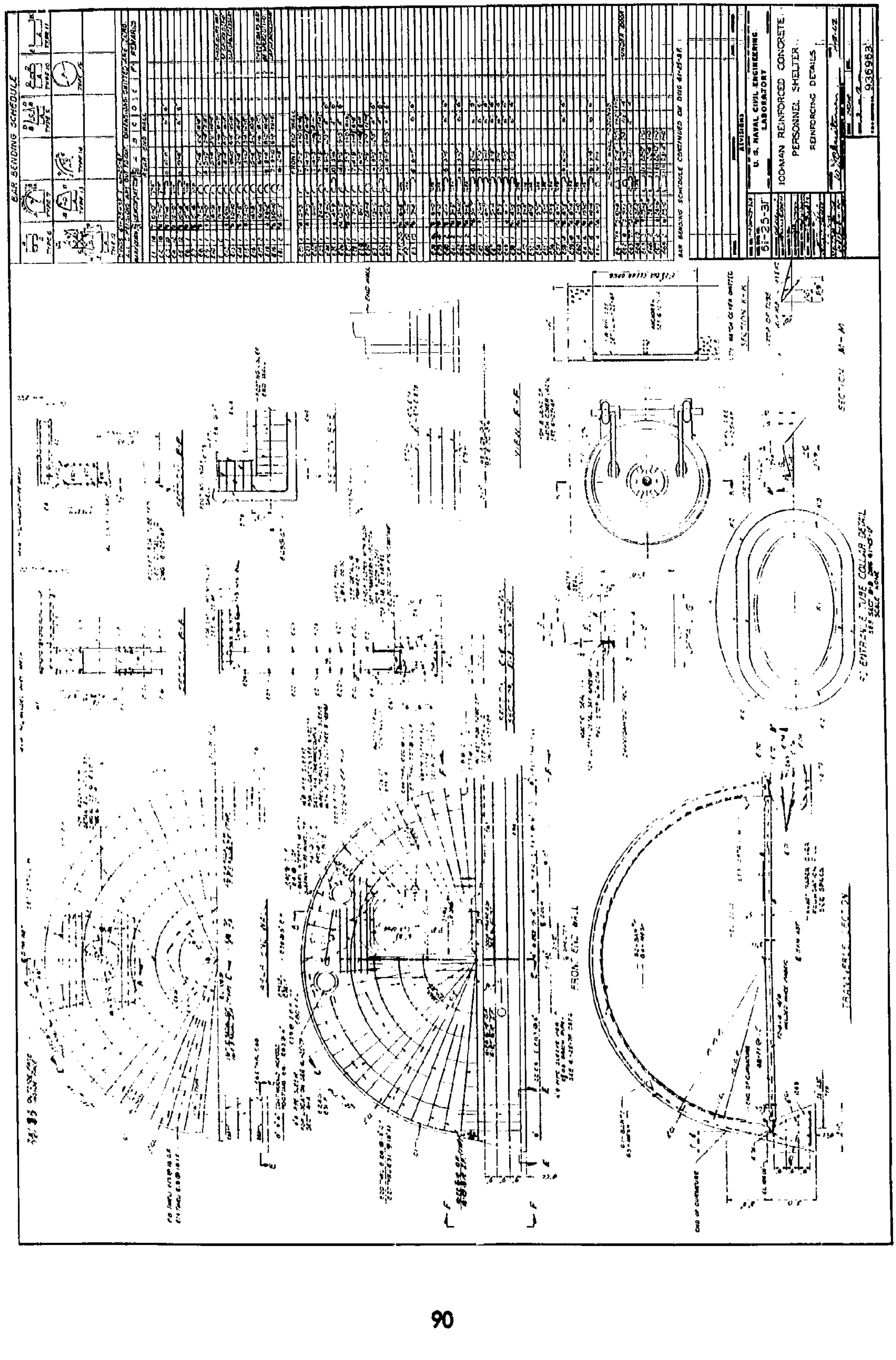
Computations	Results	Comments
Natural frequency ω:		
$\omega^2 = \frac{K_z}{m} = \frac{29.766 \times 10^3}{1,202}\ 386$	9,550	$K_z = 2k_z$ for two footings of total width 2 b
$\omega = \sqrt{9,550}$	97.5 radians/sec	
Period T:		
$T = \frac{2\pi}{\omega} = \frac{2\pi}{97.5}$	0.065 sec/cycle	
With the above values, the maximum displacement is determined from response charts (Ref. 54, p. 87) as 3.3 in. The permanent displacement would be about 2.8 in.		
This method results in deflections greater than would actually occur because the influence of rise-time of the load and arching through the soil has been neglected. Model arch tests have shown that arching is broken down on dynamic loading and comparisons of the above theory with the model tests have given conservative but reasonable results.		

Appendix E

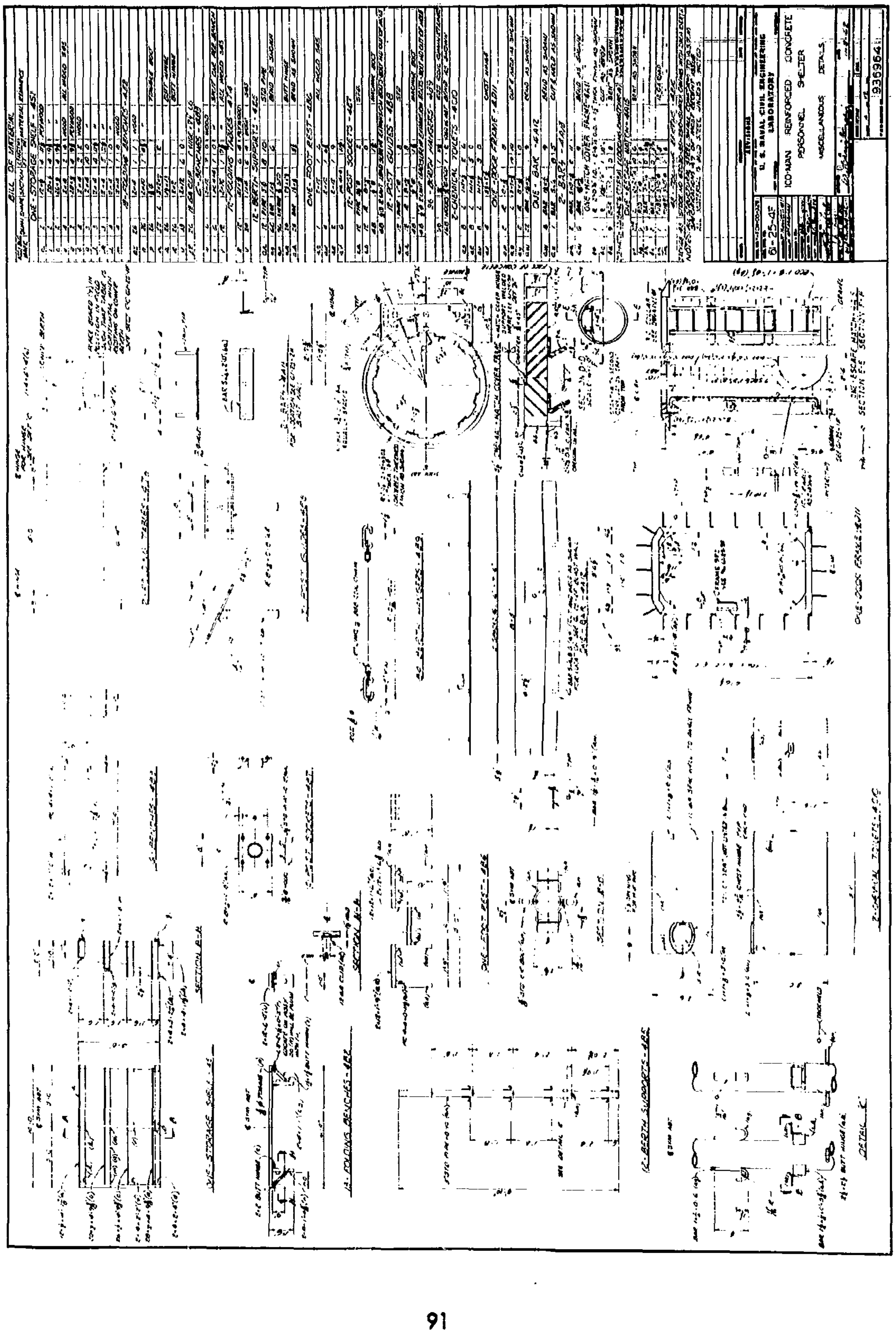
SHELTER PLANS

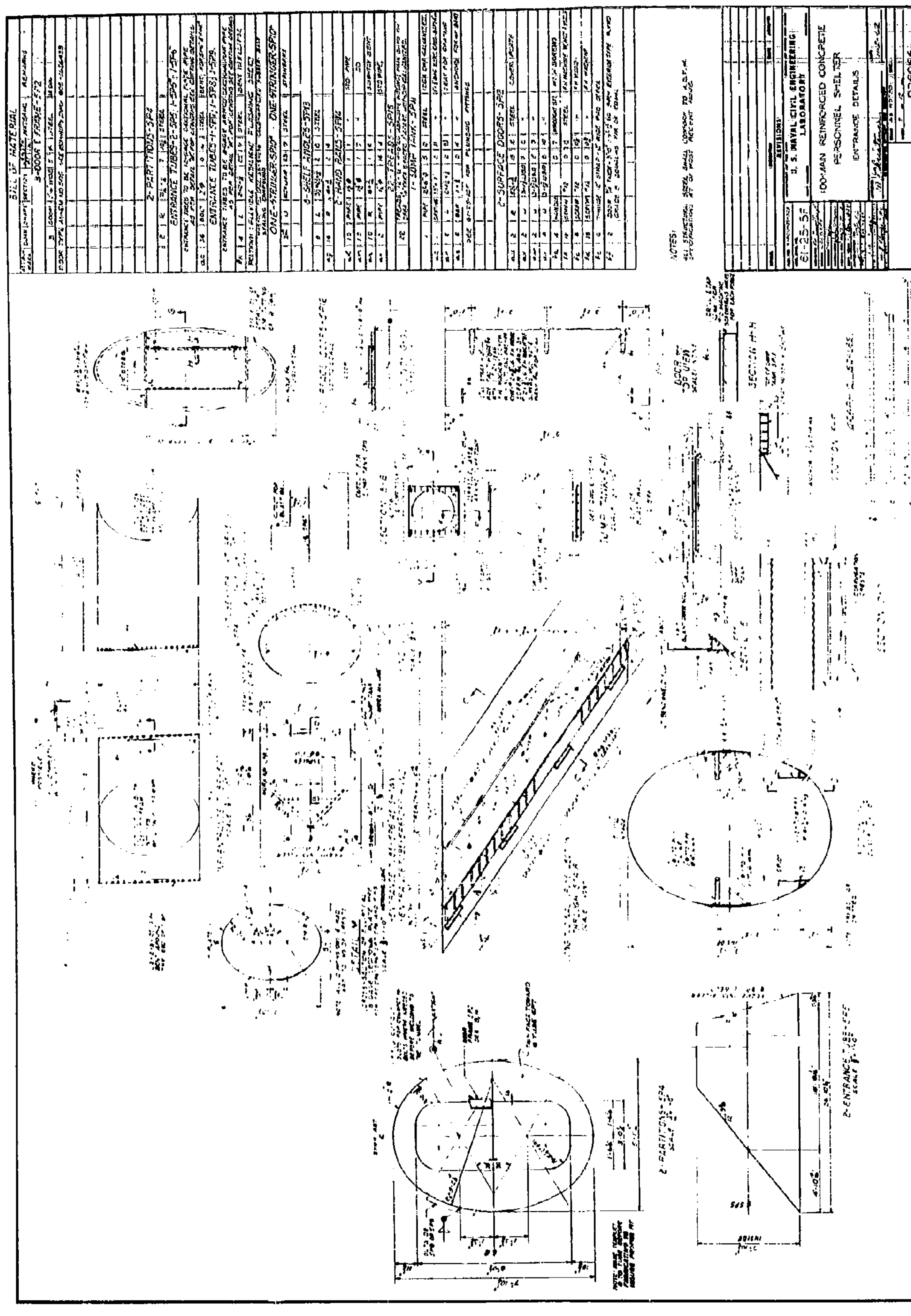

U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE
PERSONNEL SHELTER
GENERAL ARRANGEMENT
51-25-1F
936951
SECTION F-F
DETAIL H
DETAIL K
NOTES
LIST OF DRAWINGS
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION G-G
SECTION A-A

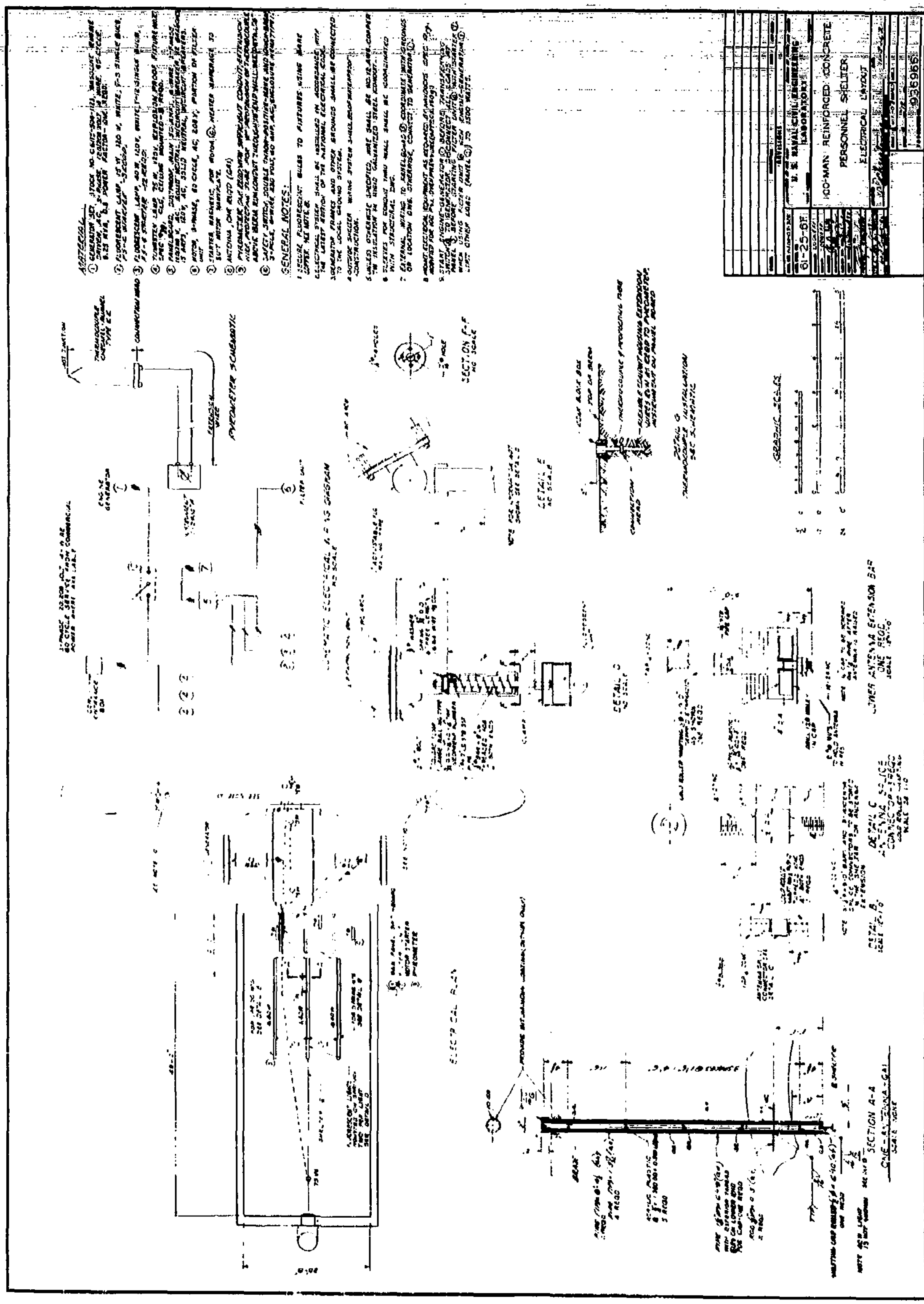

REVISIONS
U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE PERSONNEL SHELTER
REINFORCING DETAILS
935952

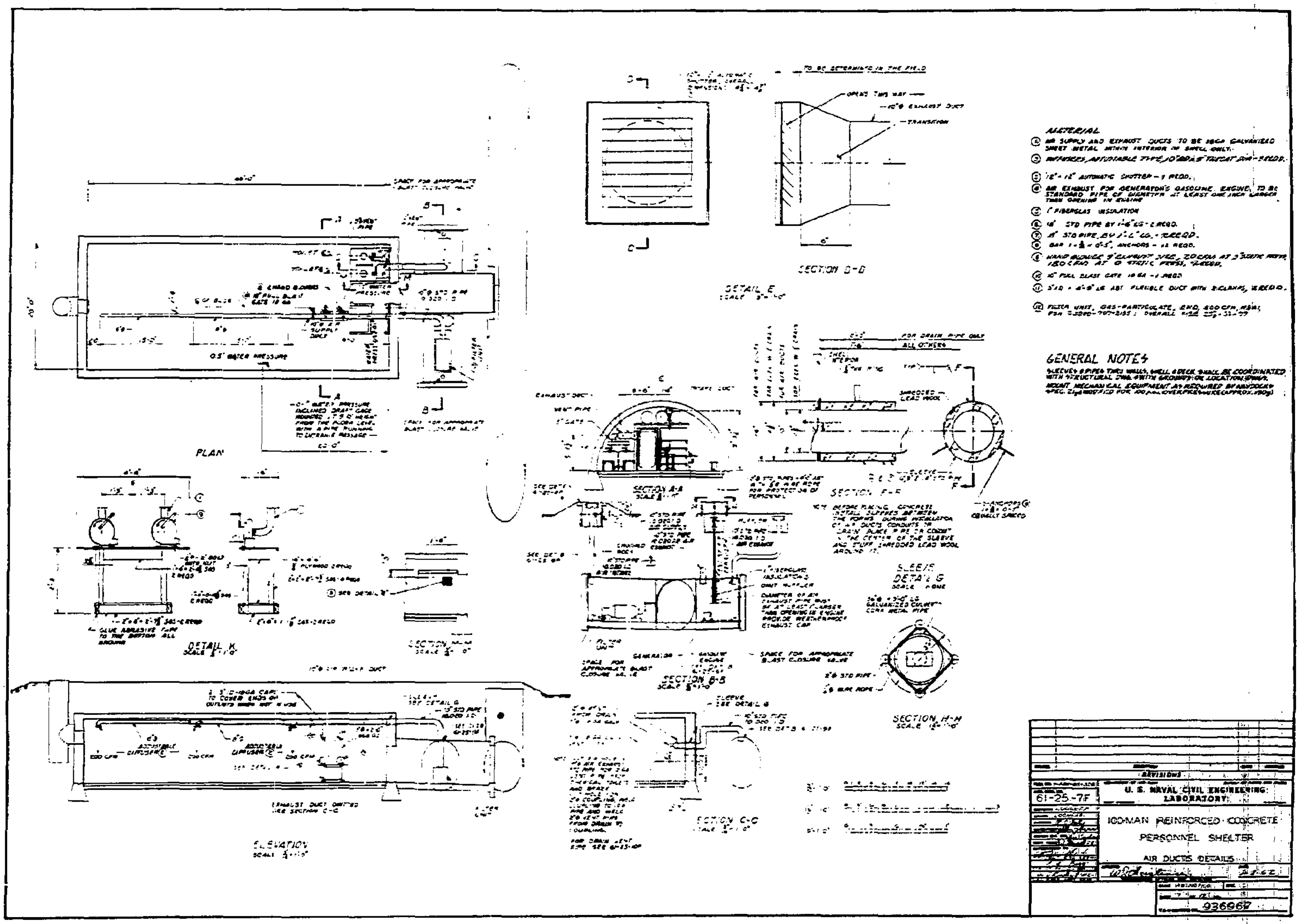


BAR BENDING SCHEDULE
U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE PERSONNEL SHELTER
REINFORCING DETAILS
936963
FRONT END WALL
TRANSVERSE SECTION

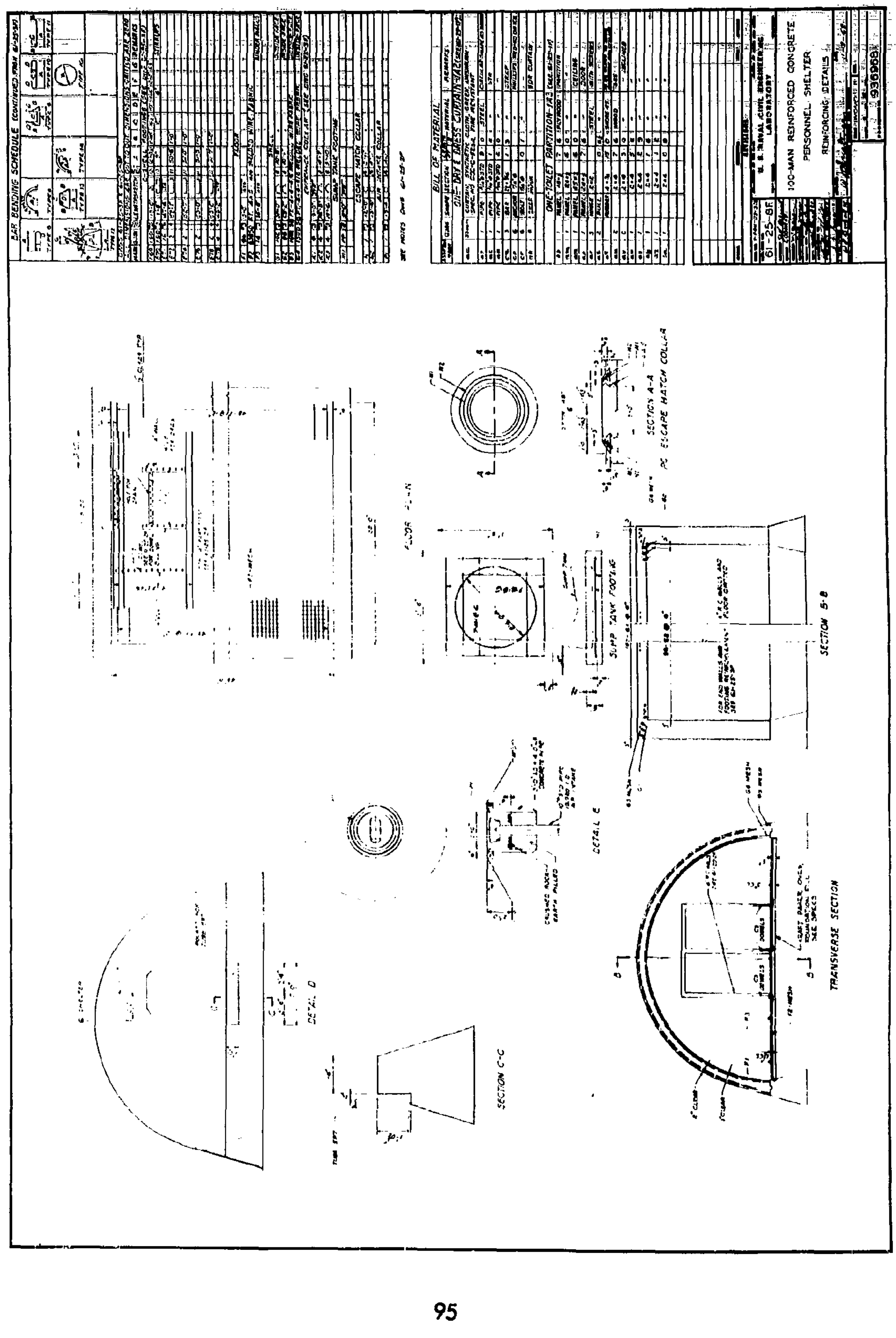

BILL OF MATERIAL
U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE PERSONNEL SHELTER
MISCELLANEOUS DETAILS
936964


U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE PERSONNEL SHELTER
ENTRANCE DETAILS
BILL OF MATERIAL

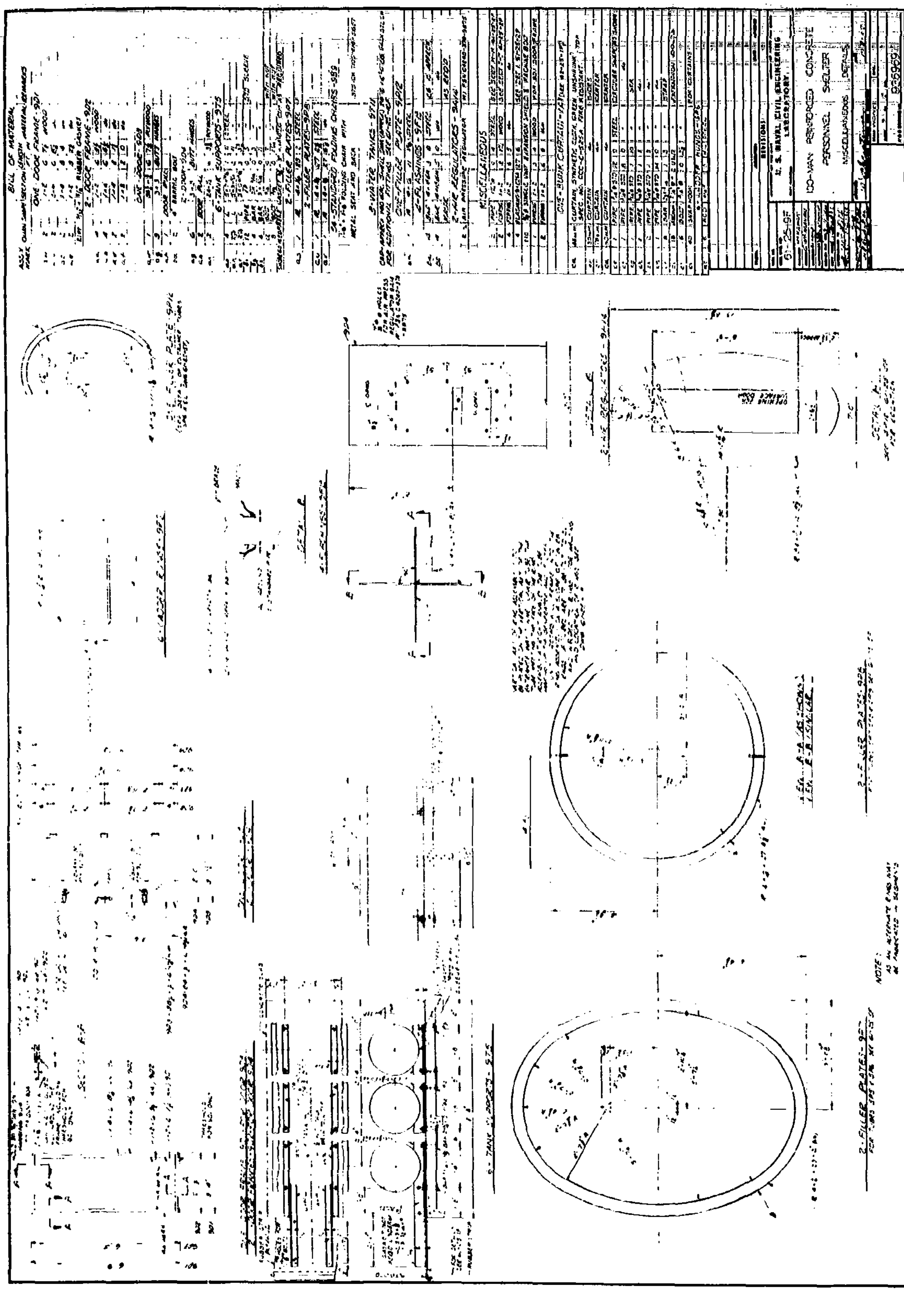

U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE PERSONNEL SHELTER
ELECTRICAL LAYOUT
936966
GENERAL NOTES:
GRAPHIC SCALE
ELECTRICAL PLAN
PYROMETER SCHEMATIC

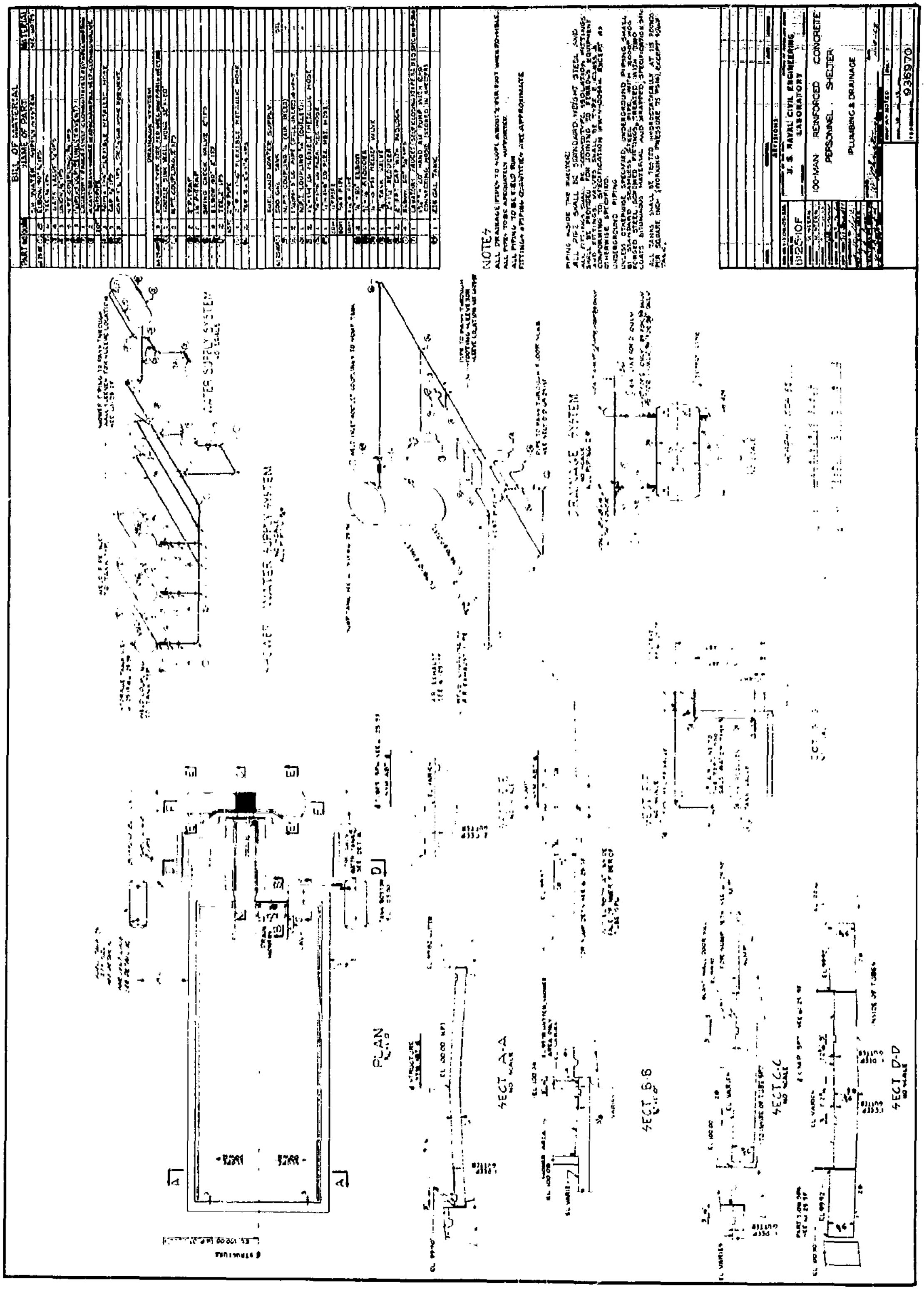


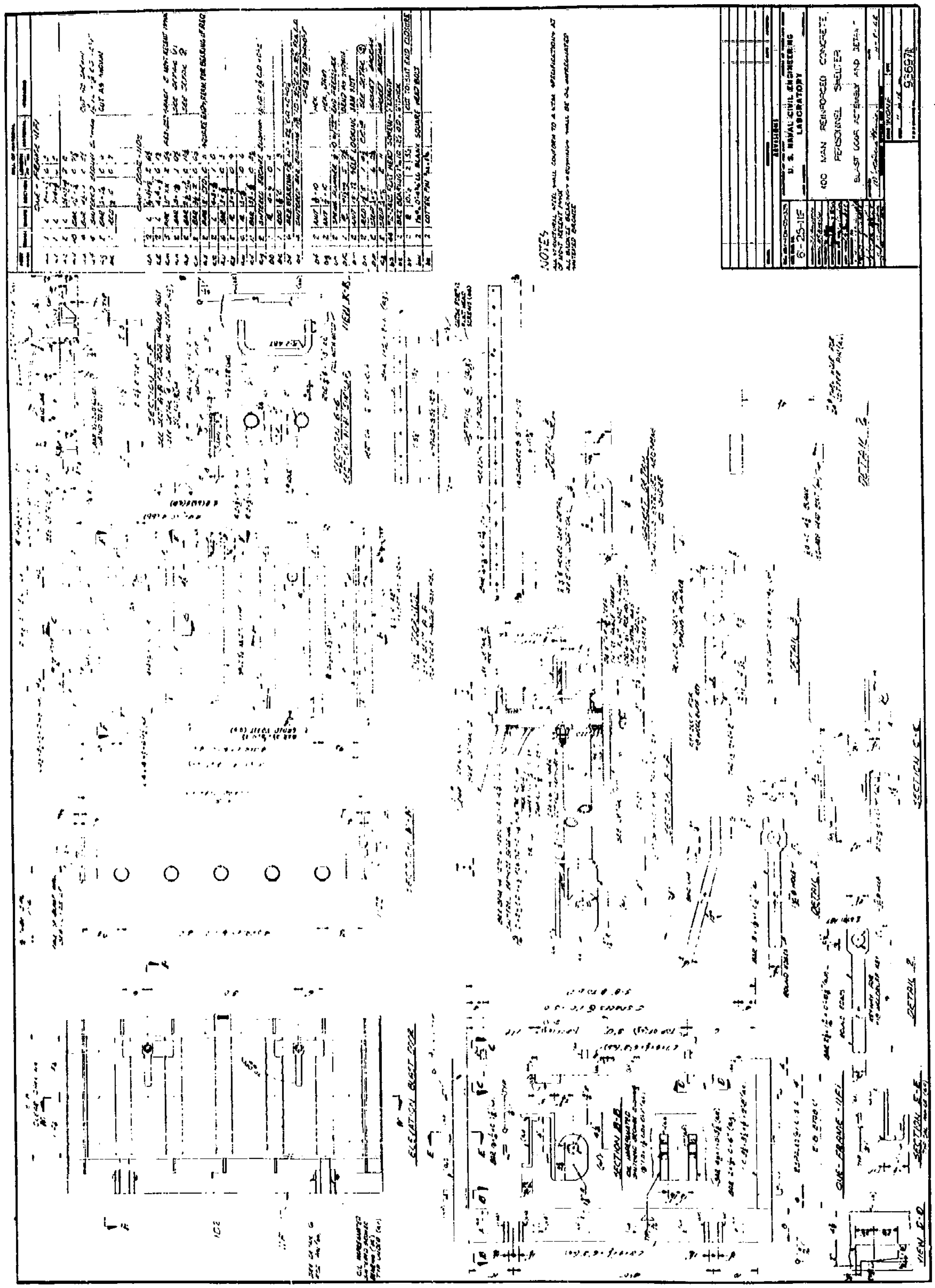
MATERIAL
FIBERGLAS INSULATION
GENERAL NOTES
PLAN
DETAIL K
SECTION B-B
SECTION A-A
SECTION C-C
SECTION D-D
DETAIL E
SECTION F-F
SLEEVE DETAIL G
SECTION H-H
ELEVATION
U. S. NAVAL CIVIL ENGINEERING LABORATORY
61-25-7F
100-MAN REINFORCED CONCRETE PERSONNEL SHELTER
AIR DUCTS DETAILS
936967

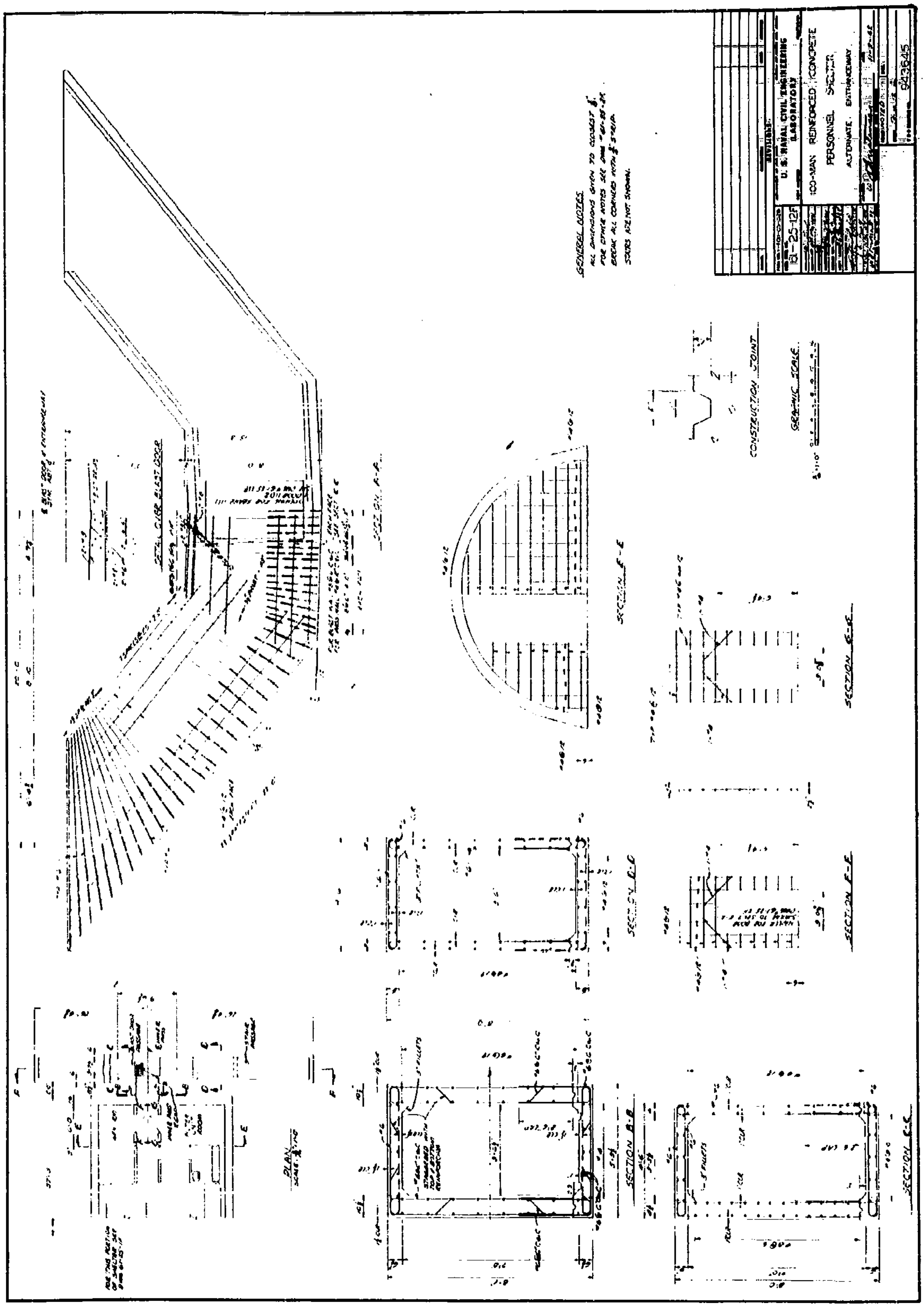


BAR BENDING SCHEDULE
BILL OF MATERIAL
U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE
PERSONNEL SHELTER
REINFORCING DETAILS
FLOOR PLAN
SECTION A-A
ESCAPE HATCH COLLAR
SUMP TANK FOOTING
SECTION B-B
DETAIL E
DETAIL D
SECTION C-C
TRANSVERSE SECTION


BILL OF MATERIAL
U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE
PERSONNEL SHELTER
MISCELLANEOUS DETAILS


BILL OF MATERIAL
NOTES
PLAN
SECT A-A
SECT B-B
SECT C-C
SECT D-D
WATER SUPPLY SYSTEM
SHOWER WATER SUPPLY SYSTEM
DRAINAGE SYSTEM
U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE PERSONNEL SHELTER
PLUMBING & DRAINAGE
936970




PLAN
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
SECTION G-G
DETAIL OVER BLAST DOOR
CONSTRUCTION JOINT
GRAPHIC SCALE
GENERAL NOTES
ALL DIMENSIONS GIVEN TO CLOSEST ⅛".
STAIRS ARE NOT SHOWN.
U. S. NAVAL CIVIL ENGINEERING LABORATORY
100-MAN REINFORCED CONCRETE
PERSONNEL SHELTER
ALTERNATE ENTRANCEWAY
61-25-12F
943645

DISTRIBUTION LIST

No. of copies	SNDL Code	
10		Chief, Bureau of Yards and Docks (Code 70)
1	23A	Naval Forces Commanders (Taiwan Only)
4	39B	Construction Battalions
10	39D	Mobile Construction Battalions
3	39E	Amphibious Construction Battalions
2	39F	Construction Battalion Base Units
1	A2A	Chief of Naval Research - Only
2	A3	Chief of Naval Operation (OP-07, OP-04)
5	A5	Bureaus
2	B3	Colleges
2	E4	Laboratory ONR (Washington, D. C. only)
1	E5	Research Office ONR (Pasadena only)
1	E16	Training Device Center
7	F9	Station - CNO (Boston; Key West; San Juan; Long Beach; San Diego; Treasure Island; and Rodman, C. Z. only)
6	F17	Communication Station (San Juan; San Francisco; Pearl Harbor; Adak, Alaska; and Guam only)
1	F41	Security Station
1	F42	Radio Station (Oso and Cheltanham only)
1	F48	Security Group Activities (Winter Harbor only)
8	H3	Hospital (Chelsea; St. Albans, Portsmouth, Va; Beaufort; Great Lakes; San Diego; Oakland; and Camp Pendleton only)
1	H6	Medical Center
2	J1	Administration Command and Unit - BuPers (Great Lakes and San Diego only)
1	J3	U. S. Fleet Anti-Air Warfare Training Center (Virginia Beach only)
2	J4	Amphibious Bases
1	J19	Receiving Station (Brooklyn only)
1	J34	Station - BuPers (Washington, D. C. only)
1	J37	Training Center (Bainbridge only)
1	J46	Personnel Center
1	J48	Construction Training Unit
1	J60	School Academy
1	J65	School CEC Officers
1	J84	School Postgraduate
1	J90	School Supply Corps

Distribution List (Cont'd)

No. of copies	SNDL Code	
1	J95	School War College
1	J99	Communication Training Center
11	L1	Shipyards
4	L7	Laboratory - BuShips (New London; Panama City; Carderock; and Annapolis only)
5	L26	Naval Facilities - BuShips (Antigua; Turks Island; Barbados; San Salvador; and Eleuthera only)
1	L30	Submarine Base (Groton, Conn. only)
2	L32	Naval Support Activities (London & Naples only)
2	L42	Fleet Activities - BuShips
4	M27	Supply Center
6	M28	Supply Depot (Except Guantanamo Bay; Subic Bay; and Yokosuka)
2	M61	Aviation Supply Office
15	N1	BuDocks Director, Overseas Division
28	N2	Public Works Offices
7	N5	Construction Battalion Center
5	N6	Construction Officer-in-Charge
1	N7	Construction Resident-Officer-in-Charge
12	N9	Public Works Center
1	N14	Housing Activity
2	R9	Recruit Depots
2	R10	Supply Installations (Albany and Barstow only)
1	R20	Marine Corps Schools, Quantico
3	R64	Marine Corps Base
1	R66	Marine Corps Camp Detachment (Tengan only)
6	W1A1	Air Station
35	W1A2	Air Station
8	W1B	Air Station Auxiliary
4	W1C	Air Facility (Phoenix; Monterey; Oppama; Naha; and Naples only)
4	W1E	Marine Corps Air Station (Except Quantico)
1	W1F	Marine Corps Auxiliary Air Station
8	W1H	Station - BuWeps (Except Rota)
1		Deputy Chief of Staff, Research and Development, Headquarters, U. S. Marine Corps, Washington, D. C.
1		President, Marine Corps Equipment Board, Marine Corps School, Quantico, Va.
2		Library of Congress, Washington, D. C.
300		Director, Office of Technical Services, Department of Commerce, Washington, D. C.

Distribution List (Cont'd)

No. of copies	
1	Chief of Staff, U. S. Army, Chief of Research and Development, Department of the Army, Washington, D. C.
1	Office of the Chief of Engineers, Assistant Chief of Engineering for Civil Works, Department of the Army, Washington, D. C.
1	Chief of Engineers, Department of the Army, Washington, D. C., Attn: Engineering R & D Division
1	Chief of Engineers, Department of the Army, Washington, D. C., Attn: ENGCW-OE
1	Director, U. S. Army Engineer Research and Development Laboratories, Fort Belvoir, Va., Attn: Information Resources Branch
1	Headquarters, Wright Air Development Division, (WWAD-Library), Wright-Patterson Air Force Base, Ohio
3	Headquarters, U. S. Air Force, Directorate of Civil Engineering, Washington, D. C., Attn: AFOCE-ES
1	Commanding Officer, U. S. Naval Construction Battalion Center, Port Hueneme, Calif., Attn: Materiel Dept., Code 140
1	Deputy Chief of Staff, Development, Director of Research and Development, Department of the Air Force, Washington, D. C.
1	Director, National Bureau of Standards, Department of Commerce, Connecticut Ave., Washington, D. C.
2	Office of the Director, U. S. Coast and Geodetic Survey, Washington, D. C.
10	Armed Services Technical Information Agency, Arlington Hall Station, Arlington, Va.
2	Director of Defense Research and Engineering, Department of Defense, Washington, D. C.
2	Director, Division of Plans and Policies, Headquarters, U. S. Marine Corps, Washington, D. C.
2	Director, Bureau of Reclamation, Washington, D. C.
1	Commanding Officer, U. S. Navy Yards and Docks Supply Office, U. S. Naval Construction Battalion Center, Port Hueneme, Calif.
1	Facilities Officer (Code 108), Office of Naval Research, Washington, D. C.
1	Federal Aviation Agency, Office of Management Services, Administrative Services Division, Washington, D. C., Attn: Library Branch
1	Director, U. S. Naval Ordnance Laboratory, White Oak, Silver Springs, Md.
1	Office of Naval Research, Branch Office, Navy No. 100, Box 39, FPO, New York
1	U. S. Naval Radiological Defense Laboratory, San Francisco
1	Officer in Charge, CECOS, Port Hueneme, Calif., Attn: ADCE Course
1	U. S. Air Force, Asst. Chief of Staff, Intelligence, Bldg. B., AHS, Washington, D. C., Attn: Mr. Sargent White
1	Commander, Air Force Ballistic Missile Division, Air Research and Development Command, P.O. Box 262, Inglewood, Calif.
1	Directorate of Research, Air Force Special Weapons Center, Kirtland Air Force Base, N. Mex.
1	Director, U. S. Army Engineer Waterways Experiment Station, P.O. Box 631, Vicksburg, Miss., Attn: Mr. G. L. Arbuthnot, Jr.
1	U. S. Army Chemical Center, Nuclear Defense Laboratory, Edgewood, Md.
1	U. S. Army Engineer School, Department of Engineering, Fort Belvoir, Va., Attn: Mr. C. D. Kepple
2	Chief, Defense Atomic Support Agency, Washington, D. C.

Distribution List (Cont'd)

No. of copies	
1	Director, Ballistic Research Laboratories, Aberdeen, Md.
1	U. S. Atomic Energy Commission, Technical Information Service, P.O. Box 62, Oak Ridge, Tenn.
1	Director, Civil Effects Test Group, Atomic Energy Commission, Washington, D. C.
1	Headquarters, Field Command, Defense Atomic Support Agency, Sandia Base, Albuquerque, N. Mex.
1	Office of the Chief of Engineers, Department of the Army, T-7, Gravelly Point, Washington, D. C., Attn: ENGNB
1	Office of the Chief of Engineers, Department of the Army, T-7, Gravelly Point, Washington, D. C., Attn: ENG MC-EB
1	Commanding Officer, Engineer Research and Development Laboratories, Fort Belvoir, Va.
1	Sandia Corporation, Box 5800, Albuquerque, N. Mex.
1	Library, Engineering Department, University of California, 405 Hilgard Ave., Los Angeles
1	Mr. William J. Taylor, Terminal Ballistics Laboratory, Aberdeen Proving Ground, Aberdeen Proving Ground, Md.
1	CAPT W. M. McLellon, CEC, USN, Office of Civil Defense Support, Bldg. T-7, Washington, D. C.
1	LTJG Edward S. Perry, U. S. Naval School, CEC Officers, Port Hueneme, Calif.
1	CAPT L. N. Saunders, CEC, USN, Commander, U. S. Naval Support Forces, Antarctica, 6th and Independence Ave., N.W., Tempo D, Washington, D. C.
1	CDP E. M. Saunders, CEC, USN, Bureau of Yards and Docks, Code 74, Washington, D. C.
1	CDR H. W. Stephens, CEC, USN, Division of Military Applications, Atomic Energy Commission, Washington, D. C.
1	LCDR Charles W. Gulick, Jr., CEC, USN, U. S. Naval School, CEC Officers, Port Hueneme, Calif.
1	LCDR W. A. Walls, CEC, USN, Defense Atomic Support Agency, Washington, D. C.
1	LCDR C. R. Whipple, CEC, USN, Bureau of Yards and Docks, Code C-440, Washington, D. C.
1	Major F. A. Verser, Jr., USA, Defense Atomic Support Agency, Washington, D. C.
1	Mr. L. Neal FitzSimons, Office of Civil Defense, Department of Defense, Pentagon - 3B265, Washington, D. C.
1	Mr. Ben Taylor, Office of Civil Defense, Department of Defense, Washington, D. C.
1	Mr. Charles M. Eisenhauer, Radiation Physics Laboratory, National Bureau of Standards, Washington, D. C.
1	Mr. O. H. Hill, Bldg. 12, Rm. 505, Radiation Physics Division, National Bureau of Standards, Washington, D. C.
1	Dr. Lewis V. Spencer, Ottawa University, Physics Department, Ottawa, Kan.
1	Mr. E. E. Shalowitz, Protective Construction, GSA Bldg. 19th and F St., N. W., Washington, D. C.
1	Mr. G. H. Albright, Pennsylvania State University, College of Engineering and Architecture, University Park, Pa.
1	Mr. A. F. Dill, Civil Engineering Hall, University of Illinois, Urbana, Ill.
1	Dr. N. M. Newmark, Civil Engineering Hall, University of Illinois, Urbana, Ill.
1	Professor J. Neils Thompson, Civil Engineering Department, University of Texas, Austin, Tex.
1	Mr. Fred Sauer, Physics Department, Stanford Research Institute, Menlo Park, Calif.

Distribution List (Cont'd)

No. of copies	
1	Dr. T. H. Schiffman, Armour Research Foundation of Illinois Institute of Technology, Technology Center, Chicago, Ill.
1	Dr. Robert V. Whitman, Massachusetts Institute of Technology, Cambridge, Mass.
1	Mr. Werner Weber, Nuclear Engineering Consultant, N. Y. State Civil Defense Commission, P.O. Box 7007, State Office Bldg., Albany, N. Y.
1	LCDR J. D. Andrews, CEC, USN, SHAPE Headquarters, Paris, France, A.P.O. 55, New York
1	CDR W. J. Christensen, CEC, USN, U. S. Naval Civil Engineering Laboratory, Port Hueneme, Calif.
1	LCDR N. W. Clements, CEC, USN, Navy Nuclear Power Unit, Fort Belvoir, Va.
1	LTJG L. K. Donovan, CEC, USN, Navy Nuclear Power Unit, Fort Belvoir, Va.
1	LTJG Clinton W. Kelly, III, CEC, USN, Bureau of Yards and Docks, Program Officer, U. S. Naval Radiological Defense Laboratory, San Francisco
1	CDR W. J. Francy, CEC, USN, Bureau of Yards and Docks, Director, Southeast Division, U. S. Naval Base, Charleston, S. C.
1	CDR C. F. Krickenberger, CEC, USN, Bureau of Yards and Docks, Code 51, Washington, D. C.
1	Dr. Harold Brode, The Rand Corporation, 1700 Main St., Santa Monica, Calif.
1	Dr. William Kreger, Naval Radiological Defense Laboratory, San Francisco
1	Dr. Hans Tiller, Nuclear Defense Laboratory, Army Chemical Center, Md.
1	Mr. Irving Gaskill, National Resource Evaluation Center, Executive Office Bldg., Washington, D. C.
1	Major Robert S. Marcum, Defense Atomic Support Agency, Department of Defense, Washington, D. C.
1	Dr. Carl F. Miller, Office of Civil Defense, Department of Defense, Washington, D. C.
1	Mr. James C. Pettee, National Resource Evaluation Center, Executive Office Bldg., Washington, D. C.
1	Dr. A. B. Chilton, Civil Engineering Hall, University of Illinois, Urbana, Ill.
1	Mrs. Shea Valley, CRTZS, A. F. Cambridge Research Center, Bedford, Mass.
1	Dr. Ronald W. Shephard, University of California, Engineering Field Station, 1301 South 46th St., Richmond, Calif.
1	LTCOL Russell J. Hutchinson, 052921, Office of the Engineer, Camp Wolters, Mineral Wells, Tex.
1	LT R. B. Reeves, CEC, USN, Field Command, Defense Atomic Support Agency, Sandia Base, Albuquerque, N. Mex.
1	Professor J. T. Hanley, Department of Civil Engineering, University of Illinois, Urbana, Ill.
1	Asst. Professor J. Silverman, Department of Chemical Engineering, University of Maryland, College Park, Md.
1	Mr. R. D. Cavanaugh, Barry Controls, Inc., 700 Pleasant St., Watertown, Mass.
1	Mr. Kenneth Kaplan, Broadview Research Corporation, 1811 Trousdale Dr., Burlingame, Calif.
1	Mr. Thomas Morrison, American Machine and Foundry Co., 7501 North Natchez Ave., Niles, Ill.
1	Mr. W. R. Perret - 5112, Sandia Corporation, Sandia Base, Albuquerque, N. Mex.
1	Mr. Lyndon Welch, Eberle M. Smith Associates, Inc., 153 East Elizabeth St., Detroit, Mich.
1	Dr. Lauriston S. Taylor, Chief, Radiation Physics Division, National Bureau of Standards, Washington, D. C.
1	Professor Herbert M. Bosch, Public Health Engineering, School of Public Health, University of Minnesota, Minneapolis, Minn.

Distribution List (Cont'd)

No. of copies	
1	Dr. Merit P. White, Civil Engineering Department, School of Engineering, University of Massachusetts, Amherst, Mass.
1	Dr Robert J. Hansen, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology, Cambridge, Mass.
1	Mr. Harold Horowitz, Building Research Institute, National Academy of Sciences, 2101 Constitution Ave., N.W., Washington, D. C.
1	Mr. Luke Vortman - 5112, Applied Experiments Division, Sandia Corporation, Albuquerque, N. Mex.
1	Mr. Richard Park, National Academy of Sciences, 2101 Constitution Ave., N.W., Washington, D. C.
1	Dr. Harold A. Knapp, Fallout Studies Branch, Division of Biology and Medicine, U. S. Atomic Energy Commission, Washington, D. C.
1	Dr. Karl Z. Morgan, Director, Health Physics Division, Oak Ridge National Laboratory, Oak Ridge, Tenn.
1	Mr. Frederick A. Pawley, AIA Research Secretary, American Institute of Architects, 1735 New York Ave., N.W., Washington, D. C.
1	Dr. David Kleinecke, Engineering Field Station, University of California, 1301 South 46th St., Richmond, Calif.
1	Dr. E. E. Massey, Defense Research Board, Department of National Defense, Ottawa, Canada
1	Dr. Joseph D. Coker, National Resource Evaluation Center, Executive Office Bldg. Washington, D. C.
1	Dr. Charles F. Ksanda, Military Evaluations Division, U. S. Naval Radiological Defense Laboratory, San Francisco
1	LT Parker Moreland, Department of Defense, Defense Atomic Support Agency, Washington, D. C.
1	Dr. George E. Pugh, Institute of Defense Analyses, Weapons Systems Evaluation Division, Washington, D. C.
1	Dr. Robert Rapp, The Rand Corporation, 1700 Main St., Santa Monica, Calif.
1	Dr. Stephen B. Withey, Program Director, Survey Research Center, University of Michigan, Ann Arbor, Mich.
1	Mr. John Auxier, Oak Ridge National Laboratory, Oak Ridge, Tenn.
1	Dr. Eric T. Clarke, Technical Operations, Inc., Burlington, Mass.
1	LT Walter J. Eager, Jr., CEC, USN, Naval Postgraduate School, Monterey, Calif.
1	LCDR C. Curione, CEC, USN, U. S. Naval Civil Engineering Laboratory, Port Hueneme, Calif.
1	LTCOL James R. Bohanan, USAF, Headquarters, U. S. Air Force, Directorate of Civil Engineering, Washington, D. C.
1	Mr. Walter Gunther, The Mitre Corporation, P.O. Box 208, Lexington, Mass.
1	LCDR R. C. Vance, Mobile Construction Battalion 11, FPO, San Francisco
1	LCDR J. C. LeDoux, Office of Civil Defense, Department of Defense, Washington, D. C.
1	LT I. D. Crowley, CEC, USN, U. S. Naval School, CEC Officers, Port Hueneme, Calif.
1	LT S. H. Mathes, CEC, USN, U. S. Naval Construction Battalion Center, Port Hueneme, Calif.
1	Dr. James O. Buchanan, Technical Operations, Inc., South Ave., Burlington, Mass.
1	Mr. Jack C. Greene, Office of Civil Defense, Department of Defense, Washington, D. C.

U. S. Naval Civil Engineering Laboratory.
Technical Report R-191.
DESIGN FOR A CAST-IN-PLACE CONCRETE SHELTER, by J. R. Allgood, R. M. Webb, and R. F. Swalley.
105 p. illus 13 Dec 62 UNCLASSIFIED

Criteria, plans and specifications are given for a 100-man shelter designed to resist 100 psi overpressure and concomitant ABC effects. Methods for designing the shelter are presented in appendixes. The methods employed are suitable for use in the design office.

1. Shelter, personnel — Design
I. Allgood, J. R.
II. Webb, R. M.
III. Swalley, R. F.
IV. Y-F011-05-328

U. S. Naval Civil Engineering Laboratory.
Technical Report R-191.
DESIGN FOR A CAST-IN-PLACE CONCRETE SHELTER, by J. R. Allgood, R. M. Webb, and R. F. Swalley.
105 p. illus 13 Dec 62 UNCLASSIFIED

Criteria, plans and specifications are given for a 100-man shelter designed to resist 100 psi overpressure and concomitant ABC effects. Methods for designing the shelter are presented in appendixes. The methods employed are suitable for use in the design office.

1. Shelter, personnel — Design
I. Allgood, J. R.
II. Webb, R. M.
III. Swalley, R. F.
IV. Y-F011-05-328

U. S. Naval Civil Engineering Laboratory.
Technical Report R-191.
DESIGN FOR A CAST-IN-PLACE CONCRETE SHELTER, by J. R. Allgood, R. M. Webb, and R. F. Swalley.
105 p. illus 13 Dec 62 UNCLASSIFIED

Criteria, plans and specifications are given for a 100-man shelter designed to resist 100 psi overpressure and concomitant ABC effects. Methods for designing the shelter are presented in appendixes. The methods employed are suitable for use in the design office.

1. Shelter, personnel — Design
I. Allgood, J. R.
II. Webb, R. M.
III. Swalley, R. F.
IV. Y-F011-05-328

U. S. Naval Civil Engineering Laboratory.
Technical Report R-191.
DESIGN FOR A CAST-IN-PLACE CONCRETE SHELTER, by J. R. Allgood, R. M. Webb, and R. F. Swalley.
105 p. illus 13 Dec 62 UNCLASSIFIED

Criteria, plans and specifications are given for a 100-man shelter designed to resist 100 psi overpressure and concomitant ABC effects. Methods for designing the shelter are presented in appendixes. The methods employed are suitable for use in the design office.

1. Shelter, personnel — Design
I. Allgood, J. R.
II. Webb, R. M.
III. Swalley, R. F.
IV. Y-F011-05-328

UNCLASSIFIED

UNCLASSIFIED